STUDIES ON EVALUATION OF PRETREATMENT UNITS AT ALLAHABAD WATER WORKS

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by

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to the

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APRIL, 1987

To

My Mother

CERTIFICATE

Certified that the work presented in this thesis entitled "Studies on Evaluation of Pretreatment Units at Allahabad Water Works" by Shri Salil Kumar Yadav has been carried out under our supervision and it has not been submitted elsewhere for a degree.

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ABSTRACT

An investigation was undertaken at Allahabad water treatment plant to evaluate the performance of its pretreatment units. The head loss and residence time in the flocculators were determined to know the velocity gradient (G). The velocity gradients were found to be 37 and 39 sec^{-1}, slightly different from the design value of 30 sec^{-1}. Tracer study using NaCl in the flocculator revealed that the completely mixed fraction of flow is only 42%.

The distribution of flow, in the feed water channel, as well as in the sedimentation tanks, was determined by estimating the velocities and deposition along its length. It was found that flow was concentrated near the middle of the channel and hence in the tank, and water was stagnant at the end reaches indicating nonuniform distribution of inflow to the sedimentation tank. In order to know the pattern and extent of deposition, sounding studies were conducted in the sedimentation tanks. The depositions along the various cables were determined to draw longitudinal deposition profile and water depth contours for both the sedimentation tanks. Tracer study in the tanks showed the existence of strong short circuiting currents, as a result of which the mean flow through times stood at 49 min and 86 min against the theoretical detention time of 2.5 hours and 3 hours for sedimentation tank No . 2 and 1, respectively. The analysis of flow curve revealed that dead space fraction was 61% and 52%; plug flow fraction 37% and 51%, and completely mixed fraction was 63% and 49% of the effective volume of sedimentation tank No . 2 and 1, respectively. Turbidity removals were observed to be more in the beginning of sedimentation tanks and there was a slight reduction of turbidity from the middle of the tanks to the outlet end. Displacement efficiencies determined for both the tanks clearly indicated the practical unutility of sedimentation tank No. 2. As an outcome of this investigation certain modifications have been recommended, which if incorporated, may lead to the better performance of the pretreatment units.

KEY WORDS: Pretreatment units, Hydraulic flocculator, Velocity gradient, Feed water channel, Sedimentation tank, Hydrographic survey, Tracer study, Flow curve, Short circuiting.

1. INTRODUCTION

Water which is so essential for the very existance of living systems, can be scourage to humanity if it is contaminated. In India two thirds of enteric diseases are due to consumption of contaminated water. It is estimated that 73 working days are lost each year due to water related diseases. It is, hence, essential to supply water safe in quality and adequate in quantity.

Though many cities in India have water treatment plants to supply potable and palatable water, their performance is far from satisfactory. Most of the plants were constructed in mid or end of 19th century. Recently some modifications have been undertaken by Jal Nigam in some water treatment plants of Uttar Pradesh (U.P.). These include providing hydraulic flocculators before the sedimentation tanks, with the financial aid from World Bank.

The water treatment plants in major towns of U.P. are more than 50 years old and their present poor performance indicates the limitation of the design, operation and maintenance. The sedimentation tanks were designed when the present theory of sedimentation was not developed, hence some drawbacks are evident in the design which resulted in their poor performance. Another reason is due to the assumption that a water treatment plant once designed and built, is immutable, and that no modifications can be made to the plant. These assumptions have been responsible for

the preservation of obsolete design features in plants as much as 50 years old and for repetition in many new plants. These features have often led to plant performance less satisfactory than would be obtained using some of the design techniques developed in the last few decades.

Recently U.P. Jal Nigam has desired that I.I.T. Kanpur may undertake an investigation to evaluate the performance of flocculators and sedimentation tanks vis-a-vis their design in KAVAL towns to illustrate the effect of design features on plant behaviour with an ultimate objective of evolving a type design, which can be used for constructing/modifying water treatment plants in other towns of U.P.

A critical appraisal of the process design is possible only after evaluating the performance of the plant constructed based on such a design. Studies conducted on number of treatment plants are not only expected to bring out lacunae in design but also to evolve a modified design procedure which may be used for the construction of treatment plants in the country.

Keeping this in mind, the present investigation was undertaken at two water treatment plants of KAVAL towns towards two M.Tech. theses in the Civil Engineering Department of I.I.T. Kanpur. The specific objectives of the present investigation are evaluating the performance of hydraulic flocculators; in terms of velocity gradient and detention time, and sedimentation basins in terms of their hydraulic efficiency and particle removal efficiency. This thesis deals with the investigation conducted at Allahabad Water Works.

2. LITERATURE REVIEW

2.1 Historical Background:

Even before 19th century, sedimentation and filtration were known for removal of solids from water to make it suitable for drinking. Sedimentation has been used for generations to clear liquors and to concentrate solids in many diversified fields, mostly on intermittant operation basis. Hazen (1904) was the first to bring out a detailed theoretical analysis of working and design of sedimentation basins on continuous operation mode. Hazen's work was followed by Camp (1936, 1946, 1953), who presented a more rational approach for the phenomenon of solid-liquid separation, and evolved design procedures for sedimentation tanks. It has been reported by Hudson (1981) that one of the earliest mixing chambers was installed to precede sedimentation by Baylis at Jackson, Missisippi (U.S.A.) in 1914. Within a decade, mixing basins became essential parts of new water clarification plants as mixing aided better settling, longer filter runs, and better filtered water quality.

2.2 Pretreatment of Raw Water:

The raw water contains objectionable colloids which are required to be removed by alum coagulation and flocculation. During the rapid mixing; after addition of coagulant, the colloidal particles are destabilized and these destabilized particles are brought together for agglomeration by slow mixing in the flocculator. After the floc has attained

a required size, it is separated from water in sedimentation tank. The various operations in pretreatment of water are discussed in the following sections.

2.3 Rapid Mixing:

Rapid mixing is that phase of coagulation, where in the coagulant by voilent agitation of the water to be treated, is quickly dispersed throughout it, thereby resulting in the formation of subvisual floc particles. In field, rapid mixing is generally accomplished by one of the following methods like mechanical mixers, venturi type meter tubes or Parshall flumes, mixed flow pumps (turbines) and hydraulic jumps.

2.4 Flocculation:

The initial increase in size of the destabilized colloidal particles is caused by Brownian motion. To increase the rate of floc formation, gentle turbulent mixing of the suspension is required and this process is termed as flocculation. The velocity gradient required for flocculating the particles as given by Camp and Stein (1943) is

$$G = \sqrt{\frac{P}{\mu}} \qquad 2.1$$

where G = root mean square velocity gradient

μ = absolute viscosity of fluid

P = total power dissipated per unit volume.

However, there must be a limit to the energy input because the floc will grow in size until the shear force set by the velocity gradient is equal to or greater than the shear

strength of the floc, thus breaking the floc into smaller particles. The expression for this is given by Miyanami et.al. (1982)

$$\frac{N_1}{N_1^o} = K.(G.t)^{1.11} \, . \, t^{-1.11} \qquad 2.2$$

where N_1 = floc concentration after any time t

N_1^o = initial floc concentration

K = constant of proportionality

t = mean residence time.

The degree of agitation varies depending upon the nature and concentration of colloids present in raw water, the type and dose of coagulant, mixing time, type of flocculator used, water temperature and type of filter installed. The non-dimensional product Gt has often been used as design parameter for flocculators, and as per Weber (1972) it should be in the range of 10^4-10^5 with G between 10-100 sec^{-1} for normal performance. However, the studies by Argaman (1970) revealed that the linear relationship between performance and the product Gt is only an acceptable approximation for the low G range. It also indicated that in addition to G, the fine structure of the turbulent field has a significant effect on the performance. Camp (1955) has suggested that best economy should result where flocculation is carried out in several stages in a series of tanks with the velocity gradient progressively decreasing as the floc particles grow in size. This procedure was first developed by Langlier and is known

as Langlier process. This process of tapered flocculation has been subject of investigation by many researchers (Bhole and Prasad, 1973).

2.4.1 Types of Flocculators:

Two general categories of flocculators are mechanically agitated units; and chambers in which baffles provide the agitation. Mechanical flocculators usually include some baffling to control short circuiting. Mechanical agitation may be classed as rotary or reciprocating. The former include reel-type units, assorted paddle design, axial flow impellers and radial flow impellers (turbines). Reciprocating units include walking beam, reciprocating agitators using either rigid blades or metallic ribbons and rotating paddles.

Hydraulic flocculators with over and under baffles or around the end baffles, have been frequently used for flocculation. Their advantages are, the absence of complicated mechanical equipment and freedom from short circuiting. By varying the spacing and configuration of the baffles, a basin can be designed for a specific mixing intensity. This can also be reduced in successive compartments to yield decreasing velocity gradients. However, these basins cannot be easily modified when mixing intensity requirements change. Some disadvantages of hydraulic flocculators over mechanical stirrers are (Camp, 1955)

1) G values are too high at the 180° bends because most of the head loss occurs there, whereas, it is not high enough in the straight channels.

2) Since velocity gradient is directly proportional to the rate of discharge, it cannot be varied at will by the operator.

However, according to Hudson and Wolfner (1967) the other advantages of hydraulic flocculators are

a) They are easier to drain and clean.

b) As head loss varies as the square of the flow to the tank, it can be designed to maintain a reasonably uniform work input for a wide range of flows. This can be achieved by designing substantial head loss, so that the depth of flow increases with quantity of flow.

2.4.2 Design Procedure for Hydraulic Flocculators:

The design procedure for hydraulic flocculators is still semi-rational, and is as following. First, suitable G and t values to give optimum flocculation as described in Section 2.4 are selected. In order to achieve the selected G value, the head loss that should be induced in the flocculator can be determined from the fundamental expression

$$G = \left(\frac{w \cdot h_f}{\mu \cdot t}\right)^{1/2} \qquad 2.3$$

where h_f = head loss to be achieved in the flocculator

w = specific weight of the water

t = detention time of the flocculator.

Knowing the discharge to the flocculator the baffles are designed to yield the requisite head loss. Further the baffle length, depth of flow and spacing can be calculated in the following steps

1) The average velocity of flow in the channel v_c (10-45 cm/sec) is assumed.

   Therefore, the length of flocculator $= v_c \cdot t$

2) The cross-sectional area of flow $= \frac{\text{Discharge}}{v_c}$.

   The depth of flow (y), same as flocculator depth, is assumed hence the channel width (b_c) is calculated.

3) The baffle slot width (b_s) is assumed and using continuity equation velocity at the baffle slot (v_s) is calculated

$$b_c \; y \; v_c \;=\; b_s \; y \; v_s$$

4) Subsequently number of baffles (N-1) is calculated using the expression

$$h_f \;=\; (N-1)\left[K_1 \frac{v_s^2}{2g} + K_2 \frac{(v_s - v_c)^2}{2g}\right] \qquad 2.4$$

where N = Number of channels

K_1 = bend loss coefficient (at 180° bend)

K_2 = expansion loss coefficient.

This is an unusual case because both bend loss and expansion loss are occurring simultaneously. This has never been reported in the literature. It has been hypothesized that the net loss will be a superposition of these two losses. For this purpose these coefficients have been introduced. These values have been taken as 1.0 and 0.5 (Surya Rao, 1980), which are cited from the literature when the bend loss and expansion loss occur separately. However, studies should be conducted on number of existing plants to study the net

effect of bend loss and expansion loss, and hence the actual values of these coefficients can be estimated.

2.5 Sedimentation:

When a discrete non flocculating particle settles, a gravitational force acts on it and a drag force counteracts it. By equating these two forces, the terminal fall velocity is obtained. This is known as Newton's law and expressed as

$$v_t = \left[\frac{4}{3} \frac{g(S_s - 1)d}{C_D} \right]^{1/2} \qquad 2.5$$

where v_t = terminal fall velocity

d = diameter of particle

C_D = drag coefficient

S_s = specific gravity of particle

g = acceleration due to gravity.

The drag coefficient as a function of Reynolds number of flow around the particle is approximated as

$$C_D = \frac{24}{N_R} + \frac{3}{\sqrt{N_R}} + 0.34 \qquad 2.6$$

where N_R = Reynolds number $\left(\frac{v_t \cdot d}{\nu}\right)$

ν = kinematic viscosity of water.

This equation is valid upto the upper limit of Reynolds number of 10^4. The laminar flow formula for terminal settling velocity is given by Stokes, as

$$v_t = \frac{g}{18} (S_s - 1) \frac{d^2}{\nu} \qquad 2.7$$

Sedimentation tanks may be horizontal or vertical flow type, with the geometry of rectangular, square or circular. As the

present investigation is concerned with horizontal flow, rectangular and some specific shaped tanks situated in treatment plants of KAVAL towns, further discussion will be confined to only these type of tanks.

2.5.1 Design Criteria for Sedimentation Tanks:

Conventional rectangular settling tanks have the following features: (1) the inlet zone; (2) the settling zone; (3) the sludge storage or sludge removal zone; and (4) the outlet zone. A brief discussion of these follows.

2.5.1.1 Inlet Zone:

Purpose of inlet is to: (1) distribute influent as uniformly as possible over the cross-section of settling zone; (2) start flow through settling zone in parallel horizontal path; (3) introduce the flow into the tank with a minimum turbulance; and (4) prevent a high velocity at the bottom of the tank where sediment is gathering and being removed. Different configurations of inlet are used in practice, such as:

i) A series of inlet pipes spaced across the end of the basin with perforated elbows.

ii) Perforated baffles.

iii) Inlet ports, discharging against baffle walls.

iv) A single deflected inlet pipe discharging below the surface against bell shaped baffles.

v) An inlet in which the flow is directed against an inclined weir deflecting the flow upward followed by the baffle at the tank surface.

vi) An inlet consisting of simple overflow weir.

vii) A return elbow discharging against the tank wall at the inlet end.

Dick (1981) has emphasized on proper design of inlet by showing the difference in removal with different types of inlet. But it is not clear, how the inlet design used in the experiment facilities can be scaled up so as to be applied in full scale plant. According to Hudson (1981), among the various baffling methods, the one successfully used has been the perforated baffle for admitting water to settling tanks. It is recommended that the port diameter should not be more than the thickness of the permeable baffle wall. It allows the vena-contracta to be formed inside the perforation, so that the jets emerge in the proper direction without causing disturbance in settling zone. The perforations can be arranged either uniformly or in zig-zag fashion

2.5.1.2 Settling Zone:

It is the zone where settling of particles take place. The design of this zone is based on surface overflow rate (S.O.R.), which is nothing but the settling velocity of a desired particle which will be completely removed in this zone. Camp (1946) has shown that performance of sedimentation basin is adversely affected, as the S.O.R. is increased. A study of El-Baroudi (1969) also confirms the generally accepted relationship between particle removal and overflow rate.

According to Camp (1953) S.O.R. varies from 0.3 to 0.7 $m^3/m^2/hr$ for primary sedimentation tanks and between 1.7 to 5.0 m/hr for secondary sedimentation tanks following activated sludge process.(Fair et.al., 1968) have stated that for plain sedimentation it may be as low as 0.24 m/hr and for sedimentation with the aid of flocculation it could be 2.93 m/hr or even higher. The general practice for horizontal flow sedimentation tanks is to provide an overflow rate from 1.25 to 1.67 m/hr while in vertical flow tanks it is between 1.67 to 2.08 m/hr (Hudson, 1981). However, as an anomaly Dick (1981) has found that within the range of S.O.R. considered in his studies (upto about 4.17 m/hr), S.O.R. had no significant effect on effluent quality.

Conventionally, long and narrow sedimentation tanks are designed, partly because of lack of knowledge of effective inlet and outlet design, partly to overcome unsatisfactory pretreatment provisions and partly to control density currents. The rule of thumb has been, to make the basin at least as long as three times its width and even six times is not uncommon. A long and narrow basin is known to reduce the liklihood of short circuiting besides minimizing the proportion of the length occupied by the turbulent inlet zone. The rectangular settling basin has a further advantage over the square one, that equipment cost can be reduced by installing cleaning mechanisms in only the first part of the basin, leaving later part for periodic manual cleaning (Hudson, 1981).

2.5.1.3 Sludge Storage Zone:

The particles removed during sedimentation are stored in this zone. The depth and configuration of this zone depends on the method and frequency of cleaning and the quantity of sludge estimated to be produced by treatment. Also this zone is profoundly affected by the behaviour of the inlet zone and outlet zone.

According to Hudson (1981), sediment deposits do not follow the removal curves with precision because of the tractive movement of the flow along the top of the sludge deposit, and because of variation in sludge density from inlet to outlet. Obviously, the denser floc particles settle first and the lighter ones later. For basins that are to be manually cleaned, a storage sludge depth of about 0.3 m near the outlet and 2.0 m or more near the beginning of the zone of settling is provided. It is convenient to slope the basin floor from the sides toward the longitudinal centreline about 10% and to provide a longitudinal slope of at least 5% from the shallow outlet end towards the deeper inlet area where the drain is normally located. Manually cleaned basins are normally cleaned hydraulically with high pressure hoses. This function may be assisted by the admission of settled water through the basin outlet.

Attempts made by designers to built basins that can be cleaned by hydraulic withdrawal of sludge through ports and manifolds in the basin floors have been generally without success. If sludge is to be withdrawn continuously or nearly

continuously from the bottom of the basin by gravity without mechanical equipment, hopper bottoms have to be used with slopes not less than 55° above the horizontal. In water, sludge will not usually move down flatter slopes under the effect of gravity.

2.5.1.4 Outlet Zone:

The purpose of an outlet device is to abstract water uniformly at the outlet end of sedimentation tank. They are placed in multiples at the effluent end of the tank and sometimes over entire surface area, in order to decrease the overflow velocity. Outlet weirs are placed to reduce actual velocity of rise and to eliminate the danger of high velocity of approach, which might induce localized high velocity rise area. Rankine (1959) has recommended some rules for weir loading according to the configuration of sedimentation tanks from ten standards.

2.6 Short Circuiting and Its Effects:

Short circuiting is observed when some part of the inflow reaches the point of outflow in less than the theoretical detention period ($\frac{V}{Q}$), and some takes much longer to do so. The primary reason for such a variation of travel time for different fluid elements is the difference in velocities and length of stream paths. Also flow distribution at the inlet and outlet zone, density and convection currents, and dead spaces in the basin are responsible for such short-circuiting (Rebhun and Argaman, 1965). The variations in turbidity, temperature and salinity of incoming water to the

sedimentation tank can induce density currents. Harleman (1961) proposed equations to compute the velocity of density currents. When turbidity of incoming water was suddenly increased from 10 to 1000 FTU, it was observed that the time taken for the density current to move from inlet to outlet zone is reduced to one tenth of nominal detention time.

Fair et.al. (1968) have quantified the basin efficiency in terms of turbidity removal due to short circuiting. However, this is not true in every case. Considering discrete particle settling in a tank with appreciable short circuiting, it is evident that the efficiency remains the same provided the average detention time and theoretical detention time remain the same. This is because the water travelling at less velocity will move more particles than designed for, and water travelling at more velocity will let less particles to settle. The two effects cancelling each other and thus basin efficiency remains unchanged. However, some reduction is seen in real basins due to turbulence and bottom scour. Short circuiting can be minimized by a judicious inlet design, supplying and abstracting water equally over the full width and depth of the basin. It can also be significantly reduced by covering the basin. This eliminates the effect of wind or heat induced currents.

2.7 Tracer Study:

Tracers are used in water treatment plants for determining the distribution of flow in tanks, basin stability and, for evaluating the hydraulic efficiency of basins.

Substances which can be used as tracers include, sodium chloride, rhodamine and rhodamine WT, fluoride compounds, lanthanum salts, lithium salts, iron - 59 and lycopoduim spores. Probably the most effective tracer substance currently is rhodamine WT, an organic compound that fluoreses intensly. It is detectable at levels of 1 μg/l in a sensitive fluorometer (Hudson, 1981).

However, when the study is to be conducted on the prototype, then the use of radioactive tracers and dyes is not feasible. The only option left is the use of common salt. Two methods of applying tracers, to determine residence time are: (i) "slug dose" in which the tracer is added all at one time; and (ii) the "step dose" in which tracer is added at a fixed rate to water during the time of investigation. Disadvantage of the slug dose particularly when NaCl is used, is that the solution of salt having more density, sinks to the bottom, subsequently diffuse slowly to yield misleading results.

For ideal plug flow tanks the concentration of tracer at outlet, should rise instantaneosly corresponding to the theoretical detention time. However, in practice, it starts appearing in the outlet much earlier and then rises gradually. If duplicate runs made under the same condition yields dispersion curves, which vary in shape, then the flow pattern in the tank is unstable.

The mean, median and mode flow through periods represent the central tendencies of the time concentration distribution of the flow curves obtained from tracer studies. Mean

flow through time of the traces is calculated from the centroid of the area under the curve

$$t_{mean} = \frac{\int C_t \cdot t.dt}{\int C_t \cdot dt} \qquad 2.8$$

A short circuiting index can be calculated using modal and mean time values (Thirumurthy, 1969). Morril has given a parameter of volumetric efficiency which is known as Morril Index.

$$\text{Index of short circuiting} = \frac{t_g - t_p}{t_g} \qquad 2.9$$

$$\text{Morril Index} = \frac{t_{10}}{t_{90}} \qquad 2.10$$

where t_g = mean flow through period

t_p = modal flow through period

t_{10} = time when ten percent of tracer has been recovered at the outlet after time of addition

t_{90} = time when ninety percent of tracer has been recovered at the outlet after time of addition.

Villimonte and Rohlich (1963) and Murphy (1964) have also suggested the use of the "Index of short circuiting" to determine the efficiency of a settling tank. One more term displacement efficiency has been proposed to evaluate the overall performance of a sedimentation tank.

An efficiency of displacement is given by

$$= \frac{\text{Flow through period}}{\text{Detention period}} \times 100$$

on the basis of data of a large number of full scale water settling tanks, Cox (1964) prescribes that this efficiency

should be greater than 30% for the basin to be of practical utility.

2.8 Evaluation of Hydraulic Efficiency:

The term hydraulic efficiency describes the detention time distribution of the fluid and the flow regime in the system. As the removal is intimately connected with the hydraulic characteristics of the system, it is essential to measure and analyse the hydraulic efficiency of the system.

Rebhun and Argaman (1965) have formulated a procedure for estimating hydraulic efficiency in terms of percentage of plug flow, mixed flow and dead space fraction using the flow curve. The fundamental equation used is

$$\frac{C}{C_o} = \exp^{-(t/T)} \qquad 2.11$$

and the final equation is

$$[1 - F(t)] = \exp^{-[\frac{1}{(1-p)(1-m)} \frac{t}{T} - p(1-m)]} \qquad 2.12$$

where C = concentration of tracer recovered after time t

C_o = **weight** of tracer added per unit volume of tank

F(t) = fraction of fluid retained in the reactor for a duration less than time t

p = fraction of active flow volume acting as plug fl

1-p = fraction of active flow volume acting as mixed f

m = fraction of total basin volume that is dead spac

t = time of recovery of tracer at any instant

T = theoretical detention time.

In this method instead of three central tendency values (mean, median and modal) as used **in** previous investigations, the entire flow curve is considered for evaluating the hydraulic efficiency.

Plotting $[1 - F(t)]$ versus t/T on semi log graph yields a straight line with a slope $\log e/(1-p)(1-m)$, for $F(t) = 0$; $\log[1 - F(t)] = 0$; and for $F(t) = 0$; $t/T = p(1-m)$. From the slope of the straight line and the value of t/T for $F(t) = 0$; m and p can be computed.

2.9 Summary:

It is clear from scanning the literature that various techniques are available for evaluating the performance of flocculators and sedimentation tanks in any water treatment plant. Some of the methods which are found appropriate can be applied to know quantitatively the performance of the units in KAVAL towns.

3. SCOPE OF STUDY

The present investigation aimed at covering the following points for evaluating the performance of pretreatment units in terms of hydraulic efficiency and turbidity removal efficiency at Allahabad Water Works.

1) Velocity determination in the raw water channel and inlet channels to flocculators, to determine the total flow and the distribution of flow to each flocculator.
2) Determination of head loss and detention time value in the flocculators to calculate the velocity gradient (G) and compare it with the design value.
3) Determination of average detention time and completely mixed fraction in the flocculation by the analysis of a flow curve obtained from tracer study, to evaluate the extent of mixing in the flocculator.
4) Determination and estimation of sludge deposition around baffles in flocculators, to know the extent and pattern of deposition.
5) Determination of velocity and sludge deposition along the length of feed water channel, to know about the distribution of flow along the width and depth of sedimentation tank.
6) Estimation of sludge deposition and turbidity removals at various points in sedimentation tanks.
7) Determination of detention time, plug flow, completely mixed, and dead space fractions by analysing the flow curve obtained from the tracer study.

4. PLANT LAYOUT

The general plant layout of the Allahabad Water Works is shown schematically in Figure 4.1. Water is being pumped from river Yamuna at Gaughat near Karelabagh. At present there are 3 numbers of intake wells at Karelabagh having 10 pumps in all. These intake wells together with pumping plants will suffice for the ultimate requirement of 2001 A.D. From the intake water is pumped to the treatment plant. The treatment plant is designed for a flow of 105 MLD but at present only 60 MLD of water is being processed. The treatment plant consists of the following units.

4.1 Raw Water Rising Mains:

Raw water is being pumped to the treatment plant through four C.I. rising mains, two of 500 mm (19.685″, 508 mm) diameter and others of 600 (23.622, 609.6) and 750 (29.5275, 711.2) mm diameters, respectively, each of about 2,620 m length. The capacity of rising mains have been estimated as 98.84 MLD and 83.6 MLD for the years 1981 and 2001, respectively.

4.2 Raw Water Channel and Alum Dosing Arrangement:

Raw water pumped from the intake is delivered by the four rising mains into a circular inlet chamber located at the south-east corner of the water works at Khusrobagh. From this chamber water passes into a rectangular tank, in which a rectangular weir is fitted for flow measurement. From the rectangular tank water flows through a rectangular masonry channel 1.8 m wide, that runs along the width of

check in figure

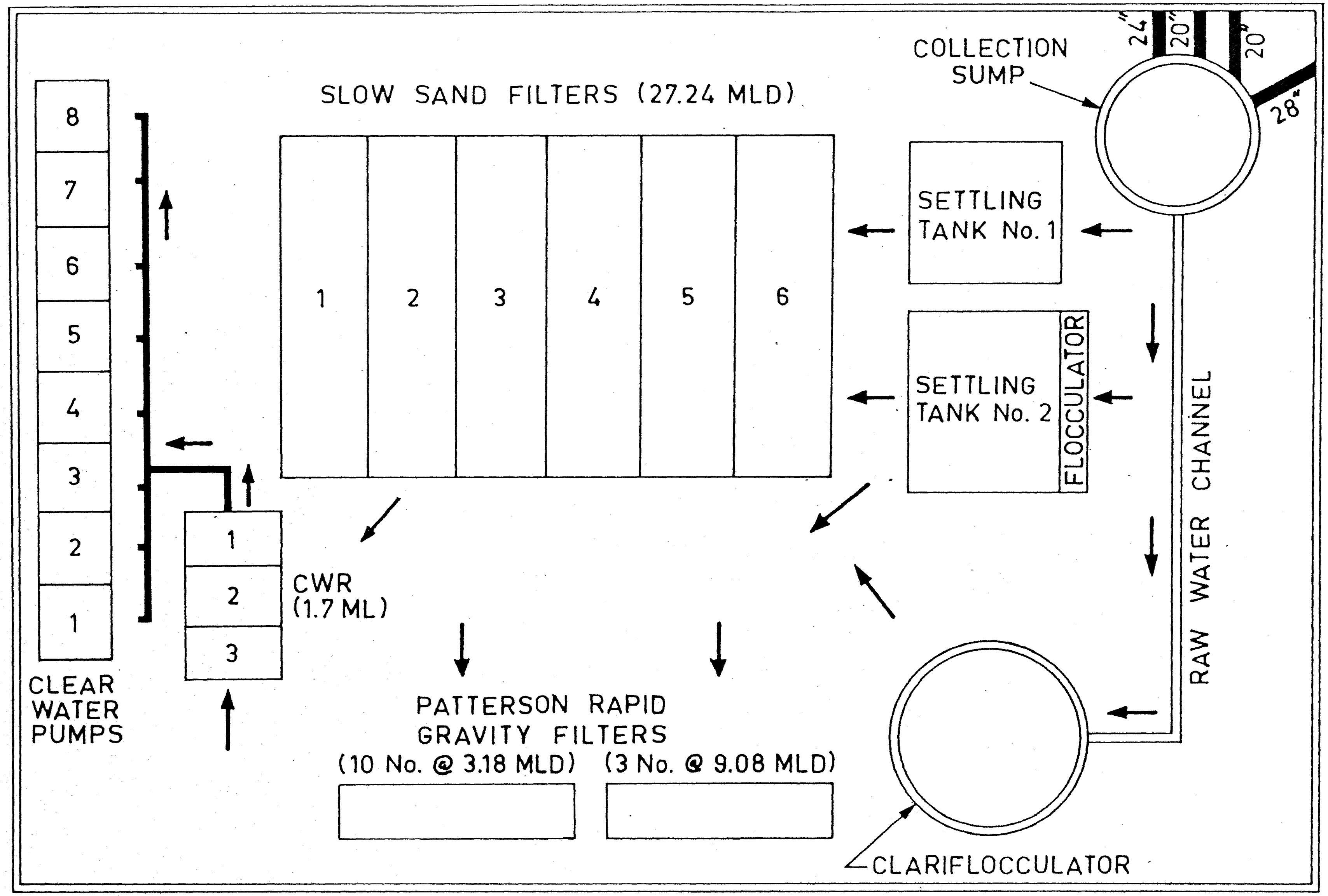


Fig. 4.1. Arrangements in Water Works Allahabad.

sedimentation tanks, shown as item No. I of Figure 4.2. This channel is termed as raw water channel. Alum solution feeding chamber is situated beside raw water channel (I) near its middle reach. The size of room is 9.0 m X 7.0 m and it can accommodate storage of dry alum and three alum dosing tanks. Each of the alum dosing tanks have 3 hours of dosing duration and can feed at 40 ppm dosage with 5% strength of solution. They are provided with an electrically operated paddle for mixing. The feeding of alum solution in the inlet channel shall be by gravity through 50 mm diameter PVC pipe and alum solution shall be disposed at the point of hydraulic jump, which is formed by steepening the slope of raw water channel. The alum feeding chamber has been comissioned but still it is out of function. So the present practice is to hang alum bricks, contained in baskets, in the raw water channel before the hydraulic jump.

4.3 Pretreatment Units:

Figure 4.2 shows the schematic layout of the pretreatment units of Allahabad Water Works. There are two sedimentation tanks out of which only one is preceeded by hydraulic flocculators. These tanks used to operate in parallel and alum was fed at one point in the raw water channel. But due to excessive sludge deposition in the tank 2, the detention time has been considerably reduced hence they are being operated in series. The tanks are connected by a bypass channel (VII). Alum is fed at two places, 1) in the raw water channel; and 2) in the bypass channel. The water works

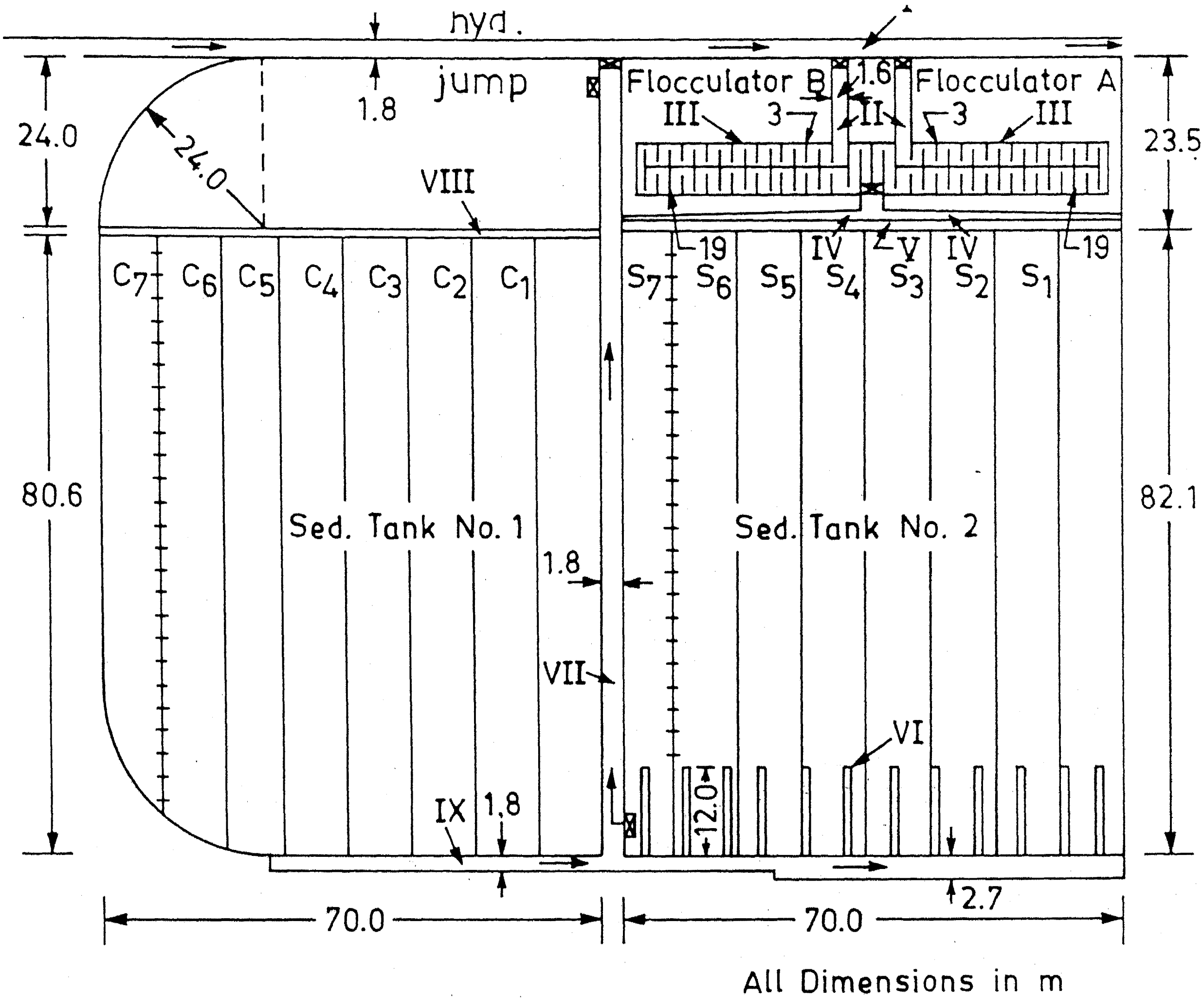


Fig. 4.2. Layout Showing Flocculators and Sed. Basins with Cables.

I	Raw Water Channel
II	Inlet Channel to Flocculator
III	Flocculator with 34 Baffles of 2.4 x 0.1 m Size at @ 1.7 m C/C
IV	Tapered Feed Water Channel
V	Perforated Wall with 0.2 m Dia. Holes
VI	Launders of 0.8 x 0.4 m Size at @ 5.83 m C/C
VII	Bypass Channel
VIII	Curtain Wall, 0.6 m Thick and with 0.1 m Dia. Holes
IX	Clear Water Channel
3	Baffle No. 3
19	Baffle No. 19

management claims some saving in alum when it is fed at these two places.

4.3.1 Flocculator (III):

Flocculators are provided for sedimentation tank No. 2 only. They are baffled type around the end flow type flocculators. Each of them is divided into two compartments. Each flocculator has 34 baffles and the size of channels is 1.60 X 2.45 m. The total channel length of flocculator is 270.0 m. The depth of flow is 2.45 m. In designing the flocculator the velocity gradient 'G' and residence time 't' have been taken as 30 sec^{-1} and 30 min, respectively. The average velocity of flow through flocculator channel has been taken as 15 cm/sec. The total head loss to be induced in the baffled channel is 14.4 cm. After flocculation water enters into the feed water channel (IV) from where it is distributed along the width of tank No. 2.

4.3.2 Inlet Arrangement:

Tank No. 1 does not have any proper inlet arrangement. The alum mixed water is directly allowed to flow in the tank through gates, while in case of tank No. 2 the flocculated water passes through a tapered feed water channel (IV). Water is allowed to enter into it near the centre and flows towards the **either** ends, as shown in Figure 4.2. The depth of the channel is 2.5 m and it is uniform, while the width varies from 1.23 m to 0.50 m. This is done to ensure uniform distribution of water into the sedimentation tank through a perforated wall (V) in the

beginning of tank. The wall is 0.2 m thick having 0.2 m diameter size circular ports, 65 in numbers and arranged in a definite pattern. This arrangement is expected to give a piston flow of water to the sedimentation basin.

4.3.3 Sedimentation Tanks:

Originally there were three sedimentation tanks, practically of the same size, each having a surface area of about 105.6 m X 70 m and a total depth of 5.30 m. They had a detention period of 3 hours and surface loading rate of about 14.2 M^3/M^2/day. One of the tank has been abandoned and in this space a clariflocculator of 45 MLD capacity has been installed. In the space, previously occupied by settling tank No. 2 flocculation-cum-settling basin has been constructed. The ultimate requirement of water is 83.6 MLD, allowing 25% extra margin, the settling tank has been designed for 105 MLD. The settling zone has been designed taking settling velocity of 1.5 cm/min, and it has a detention time of 2 hours 30 minutes. In tank No. 1 no modifications have been done. It has a curtain wall (VIII) at 24.0 m from one end of the tank, which runs across the length, and to the full depth of the tank. It has perforations of 0.1 m diameter at 1.0 m depth below the top surface. This tank has a typical shape, as shown in Figure 4.2. There is no arrangement of sludge removal in both the tanks and actual depth of tanks is reduced each year due to siltation. At present situation in tank No. 2 deposition has taken place upto the top.

width

4.3.4 Outlet Arrangement:

Tweleve launders (VI) are provided in sedimentation tank No. 2 for proper abstraction of settled water, reducing the head over the outlet channel and to check the uplifting of sludge. They are made of R.C.C., rectangular in section with size 0.8 X 0.4 m, spaced at 5.83 m c/c and have a length of 12.0 m. Head over the launders is 0.97 cm and head loss is 5.8 cm. They are supported on the R.C.C. columns. Settled water overflows from the sides of the launders and conveyed to the clear water channel (IX). Weir loading over the launders is 146 KL/MLD. In sedimentation tank No. 1, water is being abstracted over a broad crested weir which is wide running from one side to the other along the full width of the tank. Water is then conveyed to the clear water channel from where it goes to the filters.

4.4 Filtration Units:

There are two types of filters namely slow sand and rapid sand, currently in use at Allahabad Water Works.

a) Slow sand filters: Slow sand filters are six in numbers. The size of each filter is 30 X 60 m. Each filter has a capacity of 4.54 MLD and total capacity of unit is 27.24 MLD.

b) Rapid sand filters: There are 13 rapid sand filters of Patterson make of which 10 are old (6.0 X 5.4 m size) having capacity of 3.18 MLD each, and 3 are new (10.8 X 7.8 m size), having capacity of 9.08 MLD each. Filtration rates of the two types of rapid gravity filters are 3.98 $m^3/m^2/hr$ and 4.35 $m^3/m^2/hr$, respectively.

4.5 Clear Water Reservoir and Disinfection:

Filtered water from the various filters flows to the three clear water reservoirs, having total capacity of 1.7 ML. The clear water is disinfected by chlorine gas. It is done by directly passing chlorine gas into the water just before it enters into the reservoir. Finally treated water is pumped to the distribution system through clear water pumps.

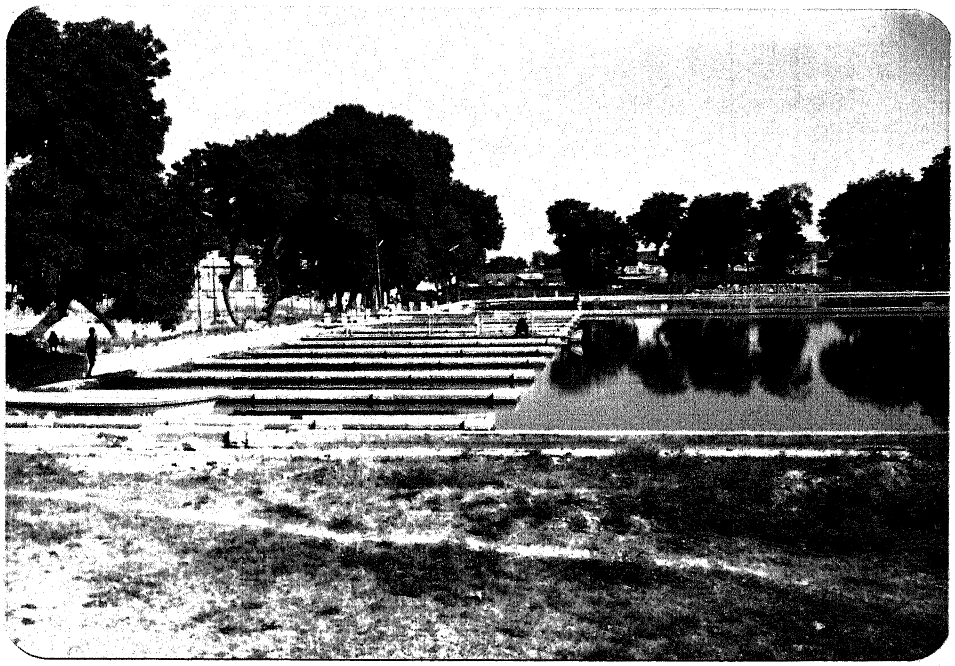
The photographs presented in the following pages represent the raw water channel where hydraulic jump is produced, flocculators, feed water channel, view of sedimentation tanks along with launders. The design details of flocculators, perforated wall and sedimentation tanks are presented in Appendix A.

1. Raw water channel, place of alum addition and hydraulic jump.

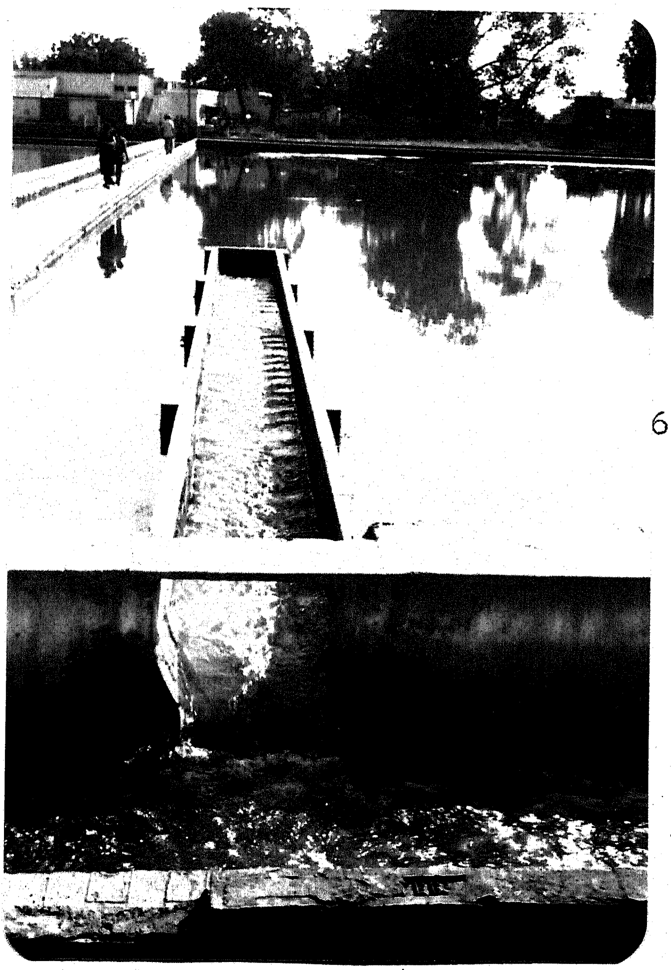
2. Inlet channels to flocculators, flocculator A and some part of sedimentation tank 2 with deposition upto the top.

3. Flocculator B, inlet and part of sedimentation tank No. 1.

4. Tapered feed water channel.



5. Arrangement of launders and boat used for sounding.



6. An active launder, clear water channel and bypass channel.

5. EXPERIMENTAL METHODOLOGY

In order to evaluate the performance of pretreatment units, which include the flocculator, inlet to the sedimentation tank and sedimentation tanks at Allahabad Water Works, the experimental technique employed both at field and laboratory level are categorised as below.

5.1 Field Studies:

The studies on existing units included velocity and head loss measurements, sounding technique and tracer studies.

5.1.1 Velocity Measurement:

Velocity of water was measured in the following units.

a) Inlet channel to flocculator to estimate the discharge in each of the flocculators.

b) In the flocculator, to determine the velocity head and in turn to calculate the head loss.

c) In the feed water channel to know the distribution of flow along the width of the sedimentation tank.

Velocity measurement was carried out using (a) current meter; (b) Pitot tube; and (c) float. The current meter (S.D. Hardson and Co., Calcutta) did not give consistent results due to malfunctioning of the electronic circuit. When the efforts to get it functioning did not materialise, Pitot tube (Dwyers, U.S.A.) was used. However, due to fluctuations in the water surface, level in the rising limb of Pitot tube was varying, so that consistent reading could

not be obtained. After considerable efforts the velocity measurements were made using the float specially designed and fabricated for this investigation. It has been discussed in detail by Hayes and Stuip (1975). The float is presented in Figure 5.1. It consists of a circular hollow ball to which the aluminium crosses are attached at regular intervals depending on the depth required. This float gives the average velocity in the channel. As the velocity distribution along the depth is not uniform, the upper layer of water makes the float move faster while the aluminium crosses attached in the lower portion offer resistance for the movement. The net result is that the float moves with an average velocity.

5.1.2 Head Loss Measurement:

As velocity in the flocculator was very less, the velocity head was neglected and water surface elevations at different points were determined using Dumpy Level (SISA, Roorkee). For this purpose a datum was selected and all the elevationswere determined with respect to that bench mark. The difference in water elevations of water at any two points is taken as the head loss between them.

5.1.3 Tracer Studies:

Tracer studies were conducted in the flocculators as well as in the sedimentation basins to estimate the flow through time and hence the hydraulic efficiency. Slug dose procedure, being easier and cheaper, was used to determine detention time distribution in tanks. As the water treatment plant is in operation, out of many tracers outlined in

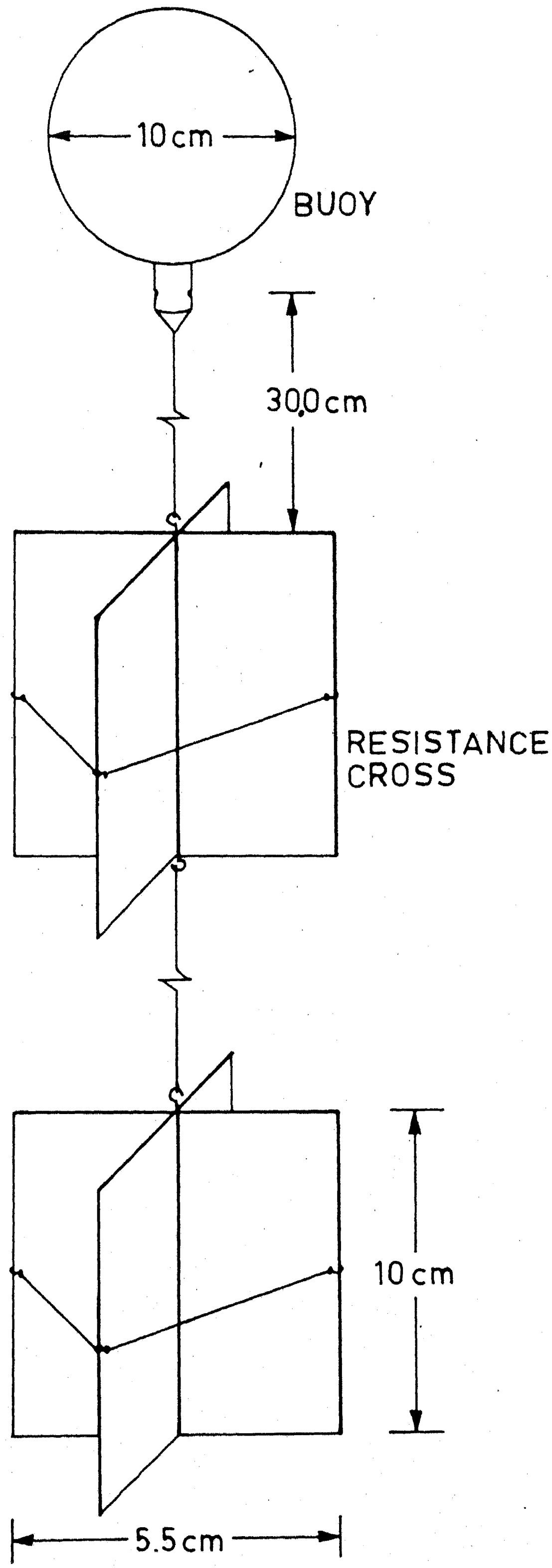


Fig. 5.1. Float Used for the Measurement of Average Velocity in the Channel.

Chapter 2 only sodium chloride (NaCl) could be used. NaCl in such quantities was added so as not to affect the water quality adversely.

After measuring the flow in the raw water channel for couple of days, enough NaCl was added to raise the concentration from the background concentration of 18 mg/l to a peak concentration of 250 mg/l. The peak was for a short time and hence its effect on finished water was negligible. For a flow of 60 MLD this worked to adding of 16 Kg of common salt for 1 min as per the procedure described by Cox (1964). The salt was dissolved in about 40 litres of water and, instantaneously applied in the feed water channel and samples were collected at different time intervals upto 4 hours. The water samples were subjected to chloride estimation as per Mohr's method (Standard Methods, 1980).

5.1.4 Deposition Profile Study:

The dead spaces in the flocculator were clearly visible by inspection of flow pattern. More dead spaces were observed near the corners. In order to quantify the sludge, the deposition around the baffles was determined. This was done using a 3.0 m long graduated plunger, at the other end of which a flange of diameter 6.5 cm was welded. The flange was allowed to rest on the top surface of sludge at various points. Using the reading on the plunger the elevation of sludge deposit at that point was determined.

It was observed that major portions of sedimentation tanks were silted up and flow pattern was irregular. It was

hence considered necessary to determine the bed configuration of the tank, so that a better analysis of tank performance can be done. For this purpose hydrographic survey was conducted and Lead line method of survey was chosen because of its simplicity and absence of sophisticated equipments. Also the depth to be measured was always less than 5.0 m. Parallel cables were run every 9.0 m apart in the tank, and across the width of the tank. Each cable had plastic balls attached at every 3.0 m distance. Thus co-ordinate systems for surface area of sedimentation tanks were generated as in Figure 4.2. At every point, sounding study was conducted by approaching the point using a boat. A plunger of 6.0 m was used with a flange of diameter 6.5 cm at one end for this purpose. The depth of sludge bed was plotted and contours joining points of equal elevation were drawn at every 0.2 m interval, to get a clear idea about bed configuration. Elevation of water tank bottom was taken as reference plane.

5.2 Laboratory Analysis :

The laboratory analysis included the turbidity measurement and chloride estimation in the water samples.

5.2.1 Turbidity Measurement:

Water samples for turbidity determination were from various points at different time intervals. The samples were collected at the points wherever sounding was done in the sedimentation tanks. Turbidity was measured in Environmental Engineering Laboratory of Civil Engineering Department at I.I.T. Kanpur using "HACH" turbidity meter

(Hach Chemical Company, U.S.A.) Model 2100A. In all, few hundreds of samples were analysed.

5.2.2 Chloride Estimation:

In tracer study, chloride was estimated in the water samples. Mohr's method was used to estimate the chloride concentration (Standard Methods, 1975)

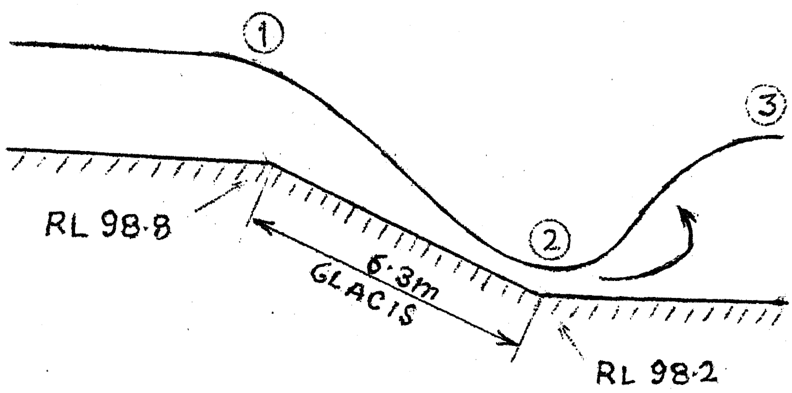


Table 6.1

Froude Number of Flow Along the Glacis

S. No.	Location of point	Froude number
1	Point 1	1.02
2	Point 2	2.70
3	Point 3	0.19

supercritical to subcritical and subsequently hydraulic-jump has been formed, which can be classified as weak jump. It was found by Iqbal (1978) that with the particular design and raw water quality, the performance of hydraulic jump is more effective at Froude numbers between 3.8 and 3.9. Alum bricks weighing about 20 Kg and containing 15-20% alum were added just before the hydraulic jump. The dissolution time for a alum brick was found to be between 2-3 hours. As the

6. RESULTS AND DISCUSSION

The results of various investigations conducted on pretreatment units for their performance evaluation, are presented in this chapter along with discussion. The sequence of results presented are identical with the sequence of units in Allahabad Water Works.

6.1 Flow Measurement:

Using the float as shown in Figure 5.1, the average velocities of the raw water in inlet channels to flocculators were determined. It was 0.3 ± 0.014 and 0.35 ± 0.02 m/sec for flocculator A and B, respectively. Based on these values the discharge of water to treatment plant was estimated to be 60 MLD. Almost same value was obtained from the reading of a bulk meter (discharge measuring device) installed downstream side of the stilling basin (referred as collection sump in Figure 4.1). This indicated that the reliable data could be obtained using the **floats.**

6.2 Rapid Mixing:

As already discussed, rapid mixing is accomplished by a hydraulic jump as shown in the following figure. For knowing the extent of mixing, Froude numbers has been calculated at various points along the glacis and presented in Table 6.1. Froude number at the beginning of the sloping glacis was 1.01, while it was 2.70 at the beginning of hydraulic jump. After the jump the Froude number has decreased to 0.19. Hence the flow is changing from

rate of dissolution varies with the surface area of alum brick, it decreases as the brick is dissolved. Hence alum dosing is not constant with time, which is expected to affect the performance of the succeeding units.

6.3 Flocculators:

The investigations carried out in the flocculators, as described in Chapter 5, have yielded the results which are presented and discussed below.

6.3.1 Determination of Velocity Gradients:

Head loss was measured using Dumpy level. It was found to be 15.0 cm and 17.0 cm in the flocculato: A and B, respectively. As per design of Jal Nigam, the head loss is 14.4 cm, so there is not much disparity.

Detention time was determined using the methods as discussed in Chapter 5. The detention time was found to be 18.5 min and 18.0 min for flocculator A and B, respectively, while the design value is 30.0 min. This reduction in detention time may be due to the deposition of sludge in the flocculatorsand hence reduction in the effective volumes.

Based on head loss and detention time values obtained, velocity gradient in the flocculators was calculated. Velocity gradient in the flocculator A and flocculator B was found to be 37 and 39 sec^{-1}, respectively, while the design value is 30 sec^{-1}. A good correlation between design and observed values is thus can be observed. The Gt values were determined based on these velocity gradients. They are 4.10×10^4 and 4.21×10^4 for flocculator A and B, however,

the design value is 5.4×10^4. As the range of G and Gt is $10-100 sec^{-1}$ and 10^4-10^5 for optimum flocculation, the values obtained at Allahabad Water Works appear to be in the desired range.

6.3.2 Flow Curve and Its Analysis:

Tracer study was conducted using sodium chloride as tracer and the flow curve for flocculator A is presented in Figure 6.1. This flow curve was analysed by Rebhun and Argaman's (1965) method to determine the flow regime in terms of completely mixed fraction. The values of F(t), (the ratio of the area of flow curve at any time to the total area of flow curve) and t/T were calculated for different time t. The plot of $[1 - F(t)]$ versus $\frac{t}{T}$ is presented in Figure 6.2. Using the slope of straight line and the value of $\frac{t}{T}$ for $F(t) = 0$, the completely mixed fraction, (1-p), was computed as 42%. In the flocculator, as mixing is essential, this fraction needs to be increased.

6.3.3 Deposition Profiles:

Estimation of deposition of sludge around some selected baffles was carried out as per the method described earlier. The deposition profile for two baffles in each flocculator, as specimen, is presented in Figures 6.3 and 6.4. The deposition around the baffle 3 in both the flocculators appears to be around 0.5 m as against the flocculator's depth of 2.45 m. The volume of the tank, thus, has been substantially reduced. It can be seen that deposition around baffle is more in the downstream side (side 'de' of

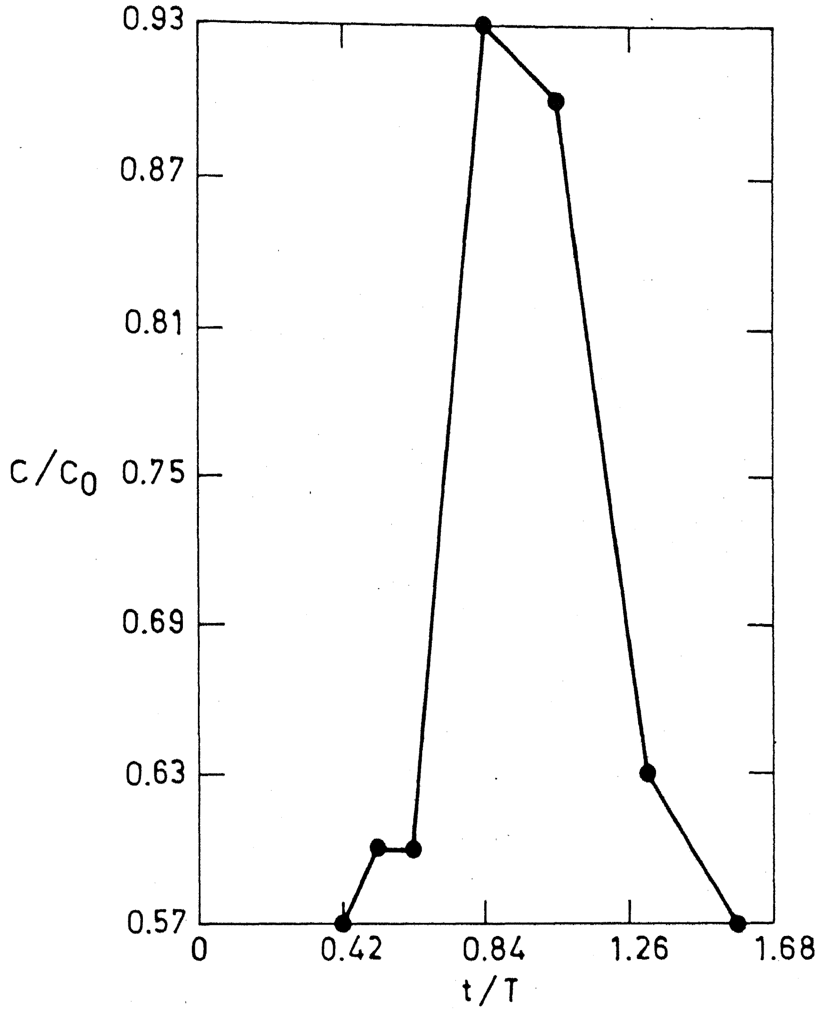


Fig. 6.1. Flow Curve for the Flocculator A Based on NaCl Tracer Studies.

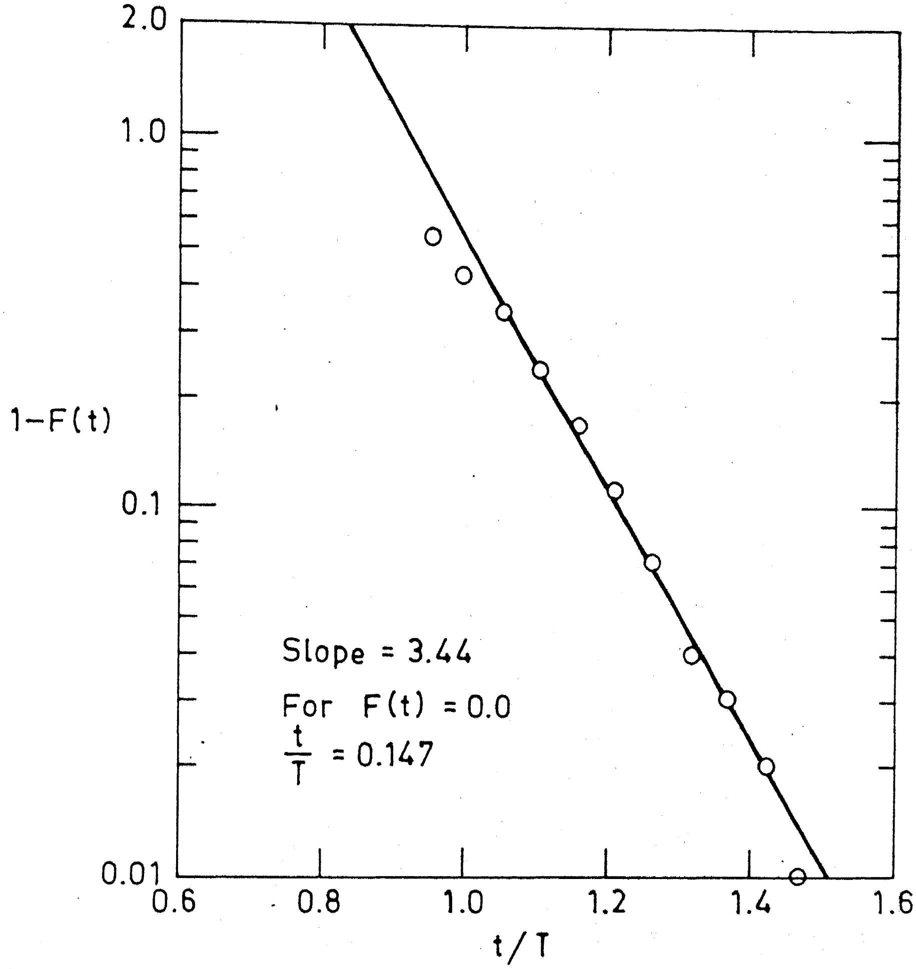


Fig. 6.2. [1−F(t)] as Function of t/T for Flocculator A.

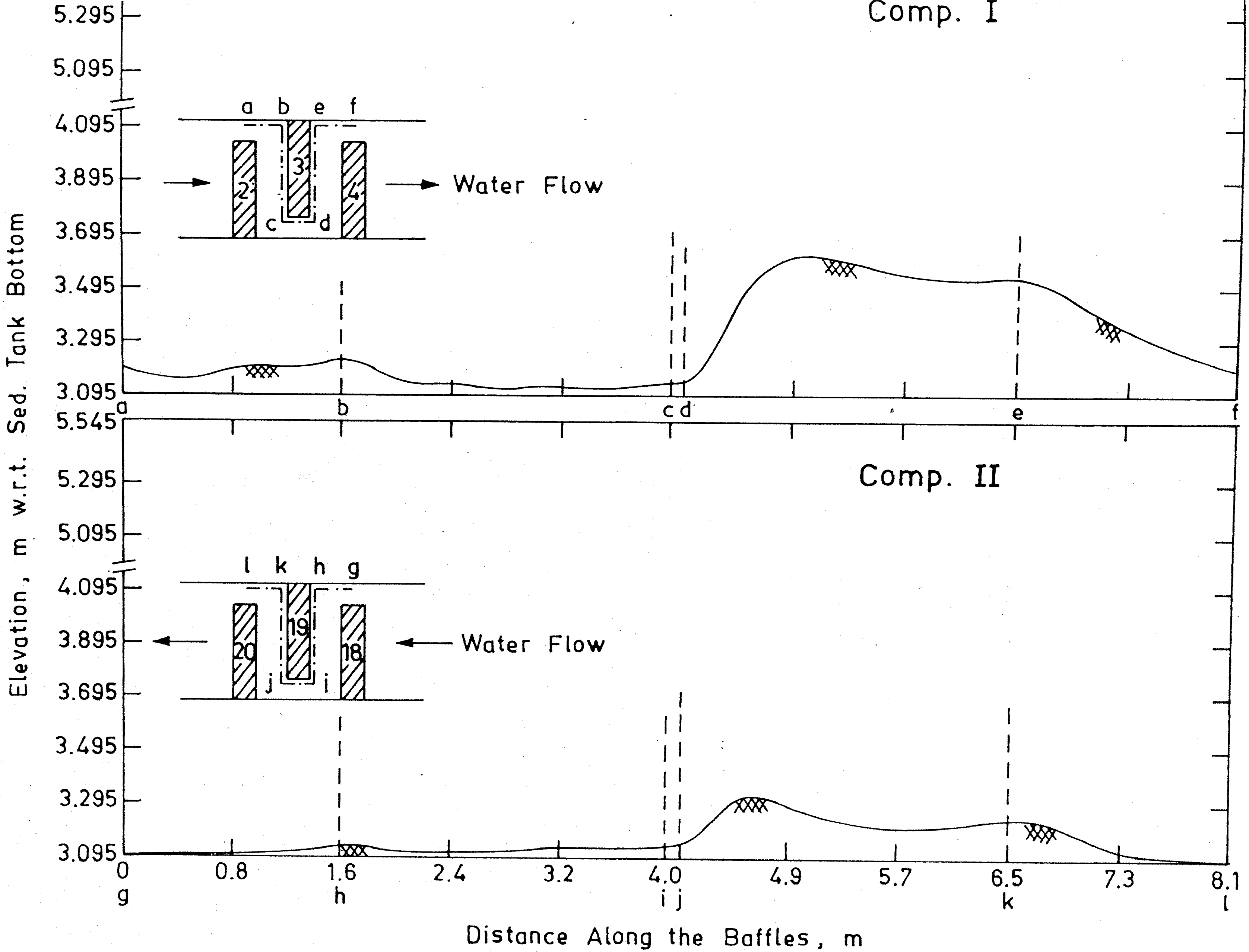


Fig. 6.3. Deposition in the Flocculator A Along Baffles 3 and 19.

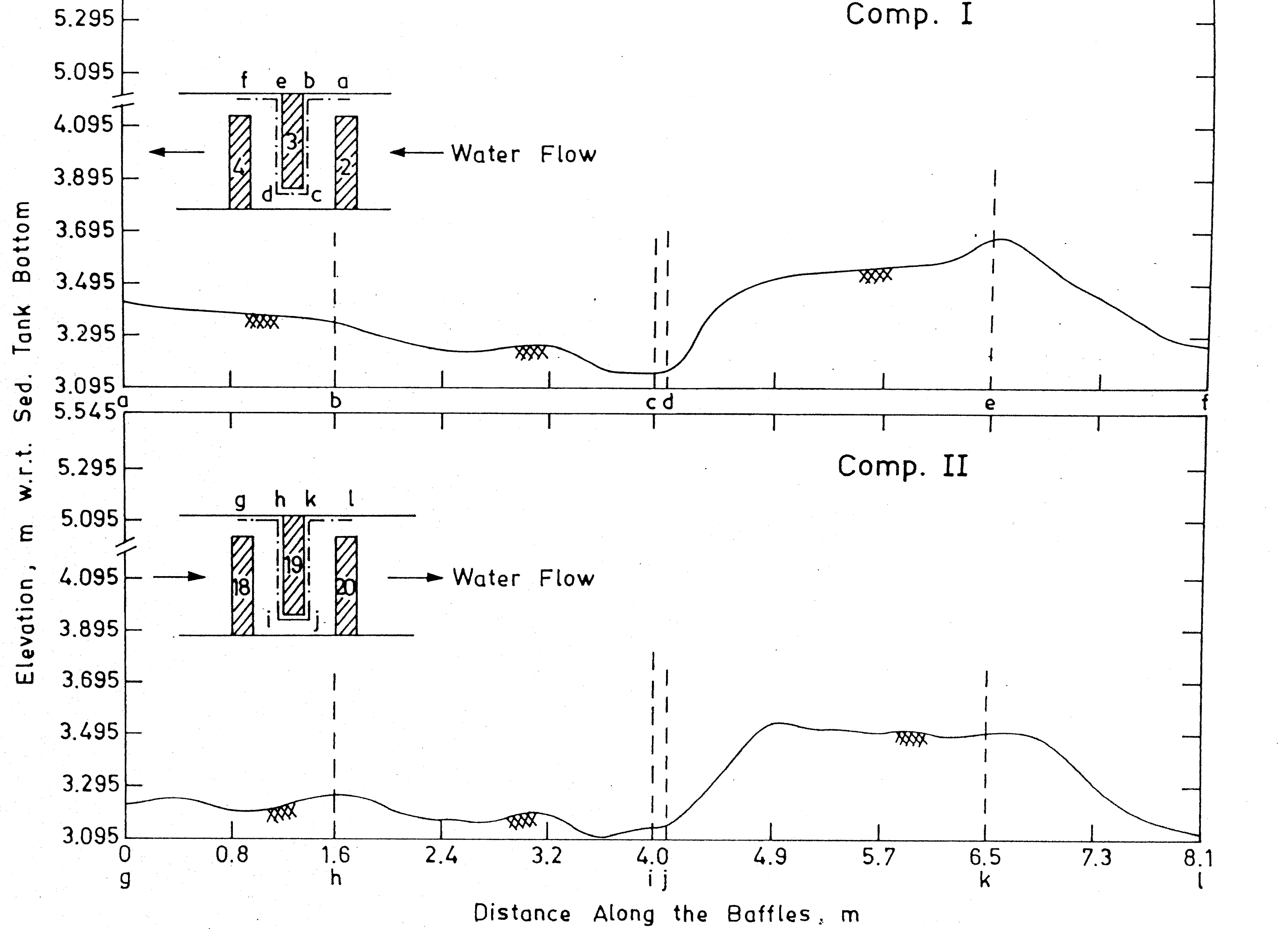


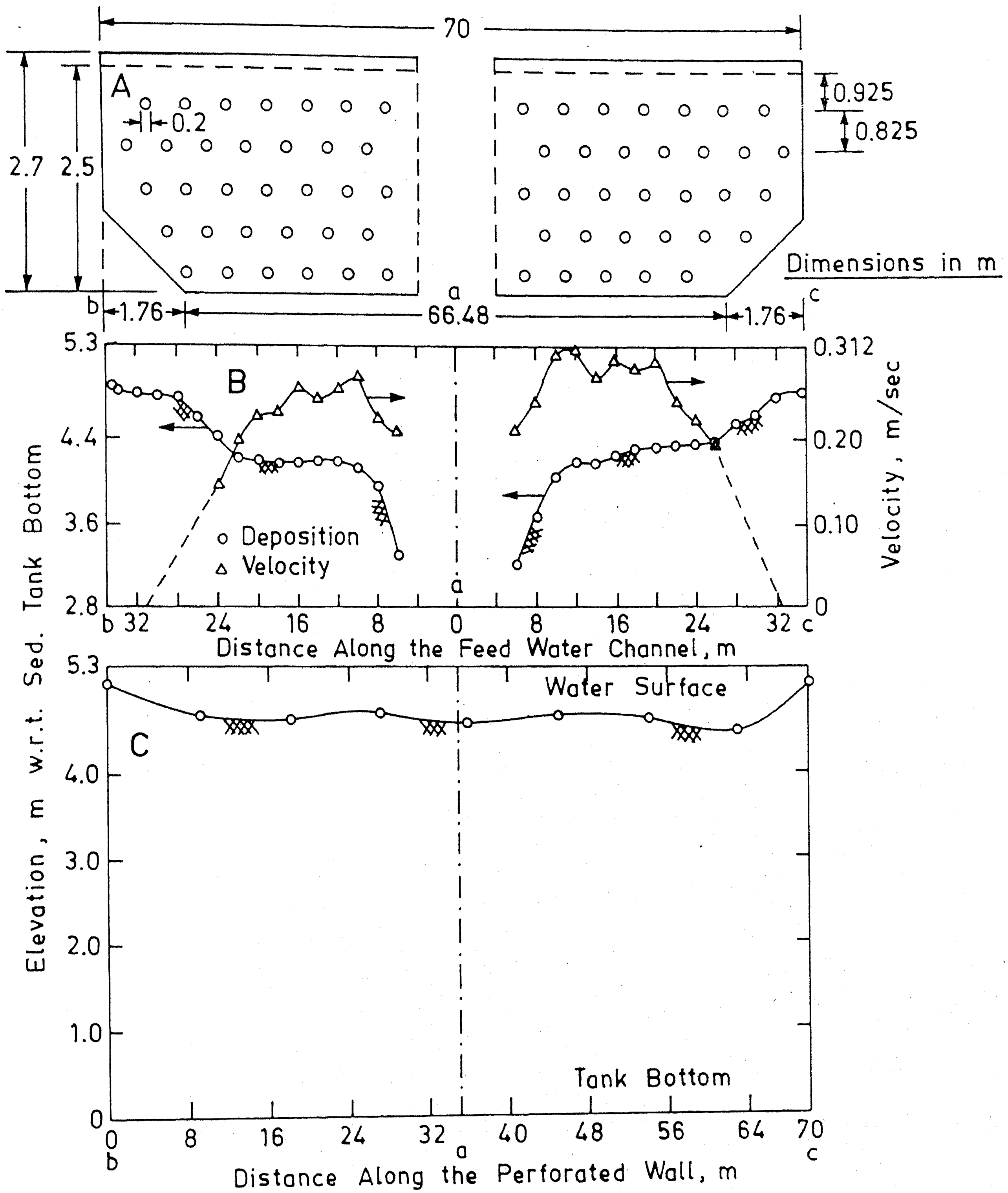
Fig. 6.4. Deposition in the Flocculator B Along Baffles 3 and 19.

the baffle) as compared to the deposition in the upstream side (side 'bc' of the baffle). It can also be seen that deposition at downstream side (corner 'e') near the corners of baffles was particularly much more as compared to the other locations. Since the flow is becoming curvilinear near the corners, due to which a part of flow is stagnant there, creating conditions for settling of flocs.

This deposition in the flocculators is not desirable as it can cause irregular flow patterns. In fact the flocs are expected to settle in the sedimentation tank rather than in flocculators. If they settle in the flocculator then the water flowing into the sedimentation tank contains flocs of smaller size which would travel a longer distance in the sedimentation tank and some of them may appear in the clear water channel at the outlet end.

6.4 Inlet Arrangement:

The details of perforated wall are shown in Figure 6.5(A). Water enters the feed water channel at point 'a' and, then flows towards point 'b' and 'c' as indicated in Figure 6.5(B). In the feed water channel the deposition of sludge and velocity distribution were obtained, and are presented in Figure 6.5(B). It is evident from the figure that more than 2.0 m of silting has occurred in the end reaches of the feed water channel as against the total depth of 2.5 m, and there the velocity of water has reduced considerably. In fact is was not possible to determine the velocity of flow after 24.0 m on either side of feed water channel, due to non movement of



g. 6.5. (A) Details of Permeable Wall (not to scale).
(B) Deposition Profile and Velocity Variation in the Feed Water Channels.
(C) Deposition Profile Along the Perforated Wall in Sedimentation Tank No. 2.

the floats. Velocity curves have been extrapolated to know the point at which stagnation of water has occurred. They are at 31.0 m and 32.0 m in the portion 'ab' and 'ac', respectively. At these points, due to deposition, the top perforations have been blocked resulting in no flow in later reaches. This reveals the nonuniform distribution of flow through the perforated wall. This can also be justified by depicting the deposition of sludge along the perforated wall as shown in Figure 6.5(C), which clearly indicates that flow is not uniformly distributed along the width, and the depth, of settling tank.

6.5 Sedimentation Tanks:

The sedimentation basins at Allahabad water treatment plant are operating in series, presently. Sedimentation tank No. 1 is downstream of sedimentation tank No. 2, hence in the same sequence their results are also presented.

6.5.1 Deposition Profiles:

The measurement of sludge depositions, obtained by sounding along the cables, have been plotted. The deposition profiles along the various cables in sedimentation tank No. 2 are shown in Figure 6.6. It is evident that the tank is silted to the top, at around 6.0 m from the perforated wall. Further the deposition is almost constant throughout the width of the tank. This is similar to the case of a reservoir, in which when a river enters then all the sediments are dropped on the reservoir bed in the beginning itself. In later reaches of the tank, the deposition

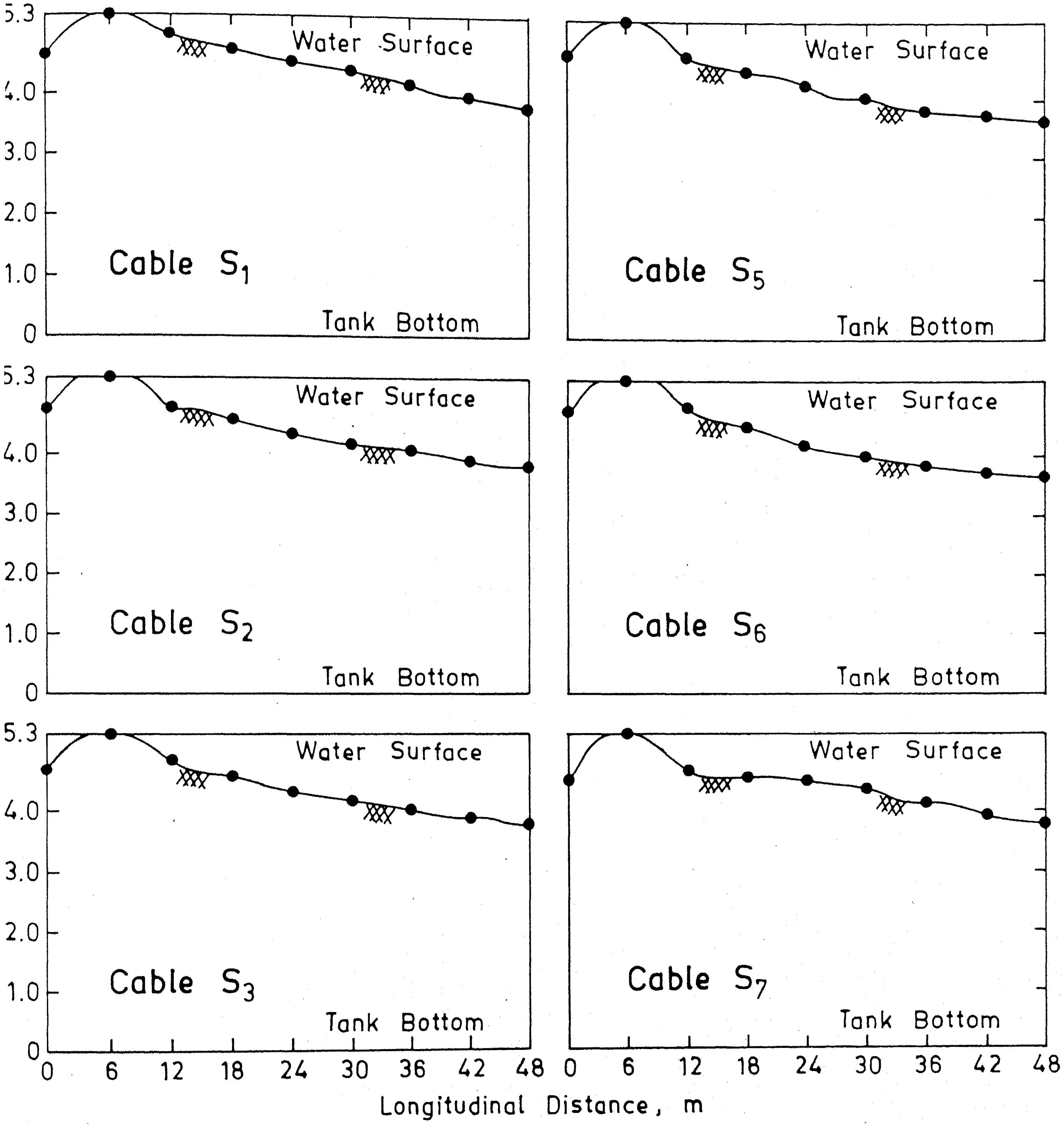


Fig. 6.6. Deposition Profile Along the Cables in Sedimentation Tank No. 2.

gradually decreased along the length of the tank. Even at 48.0 m the deposition was almost 4.0 m, as against the sedimentation tank depth of 5.3 m, beyond which it was almost constant upto the end of sedimentation tank. The effective volume has been estimated from the deposition profiles using trapezoidal method. In this method the ordinate (depth) points, at regular intervals, are joined by straight lines, assuming the top surfaces to be planes. The area of trapezoids along different cross-sections at regular intervals is calculated. Using these areas the total volume of deposit is estimated. It is estimated that volume of sedimentation tank No. 2 has been reduced to 6160 m^3 as against the tank volume of 27456 m^3, indicating that 78% of tank volume is silted.

After sedimentation tank No. 2, water flows through a bypass channel situated between the two tanks as shown in Figure 4.2, and then enters in the sedimentation tank No. 1 through a rectangular gate. The deposition profiles along the various cables in sedimentation tank No. 1 are shown in Figure 6.7. In this tank also, the deposition is more in the beginning of tank while it is less at the end of tank. But the extent of deposition is much less as compared to sedimentation tank No. 2. The effective volume has been estimated from the deposition profiles using trapezoidal method. The volume of sedimentation tank No. 1 has reduced to 18360 m^3, while it was 35715 m^3. Hence 49% of volume has been silted up.

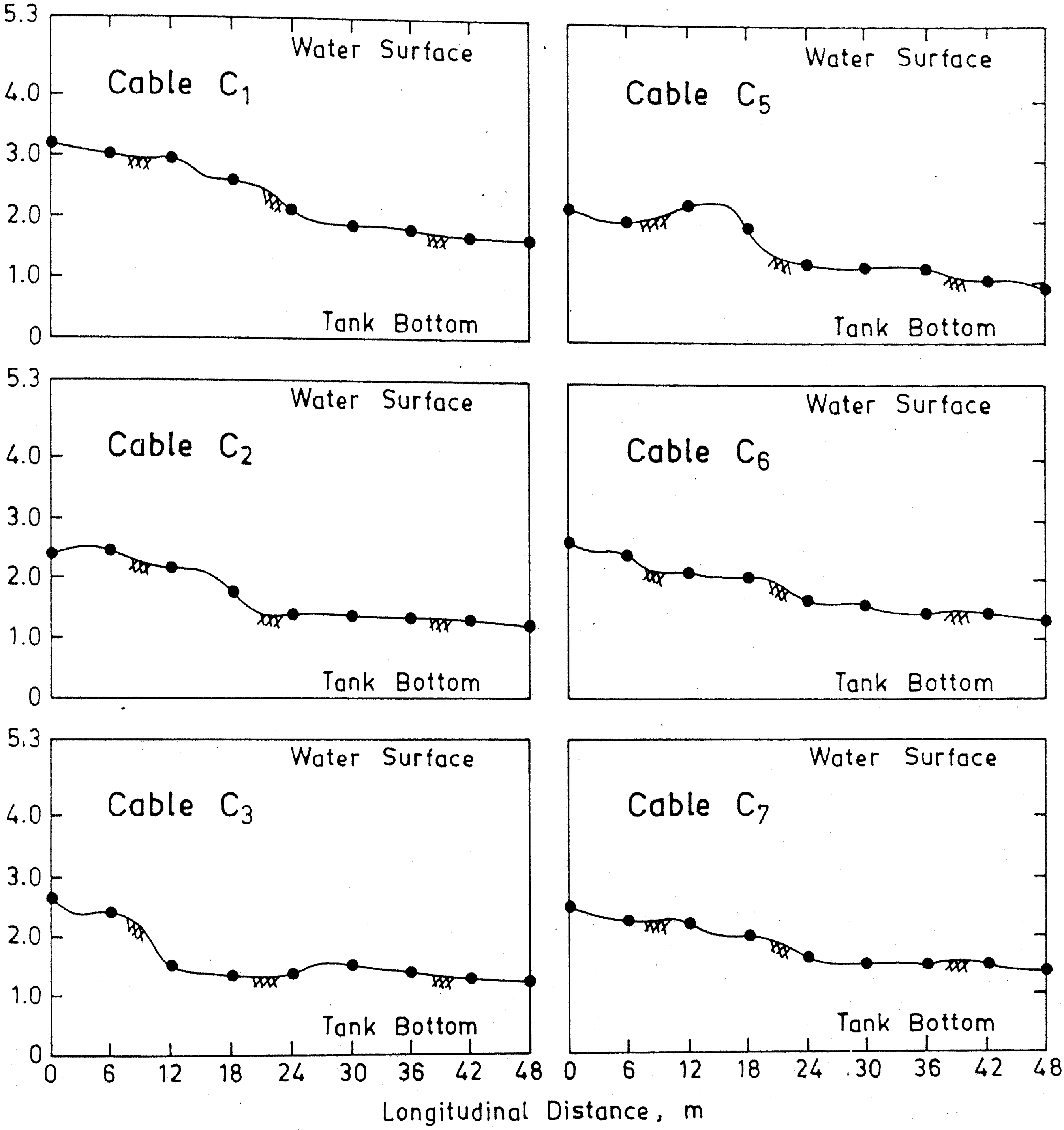


Fig. 6.7. Deposition Profile Along the Cables in Sedimentation Tank No. 1.

Water depth contours have been drawn at an interval of 0.2 m, using the method of interpolation for sedimentation tank No. 2. The contour map presented in Figure 6.8 reveals that the deposition is stepping down in the direction of flow, similar to terrace formation by rivers. Further, a steep cross slope from the sides of the tank towards the longitudinal centre line is also evident. The contour map for sedimentation tank No. 1 presented in Figure 6.9, is significantly different. Besides a less steep cross slope from the sides of the tank towards the longitudinal centre line, island formation is also observed which reduces the surface area.

6.5.2 Flow Curve and Its Analysis:

The flow curves for both the tanks, obtained from tracer study using slug dose method are presented in Figure 6.10.

The tracer first appeared at the outlet of sedimentation tank No. 2 in only about 18 min, while the design detention time is 2.5 hours, which indicates occurrence of short circuiting. This may be due to the fact that water is being abstracted over a very short length of a rectangular weir in the left side of tank as shown in Figure 4.2, and the launders being plugged. The excess deposition caused the water to travel along various channels formed in the tank bed. In sedimentation tank No. 1 also, short circuiting was observed since the tracer arrived at the outlet in only about 20 min, while the design detention time is 3 hours. This short

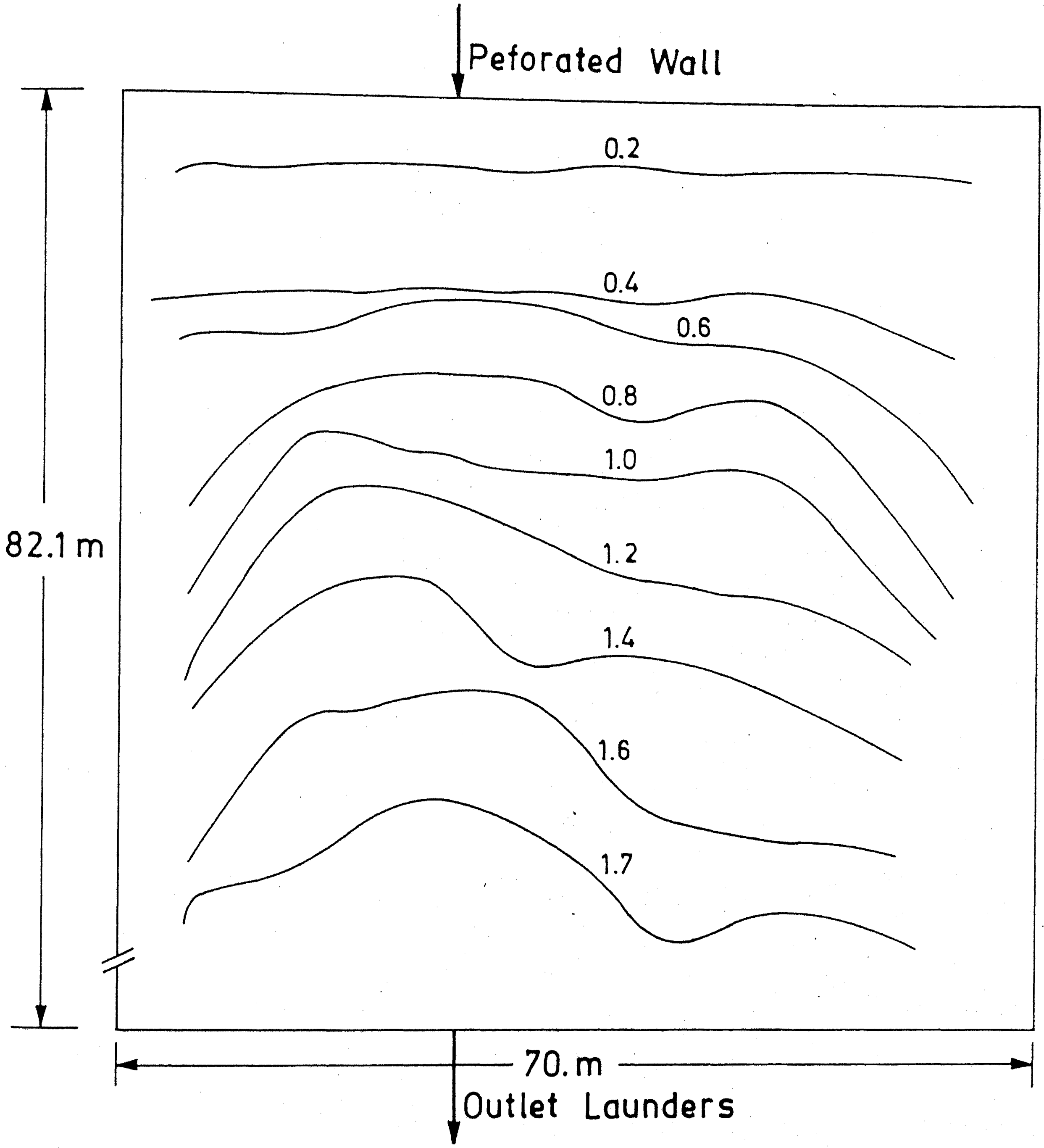


Fig. 6.8. Water Depth Contour Map of Sedimentation Tank No. 2 from Hydrographic Survey .

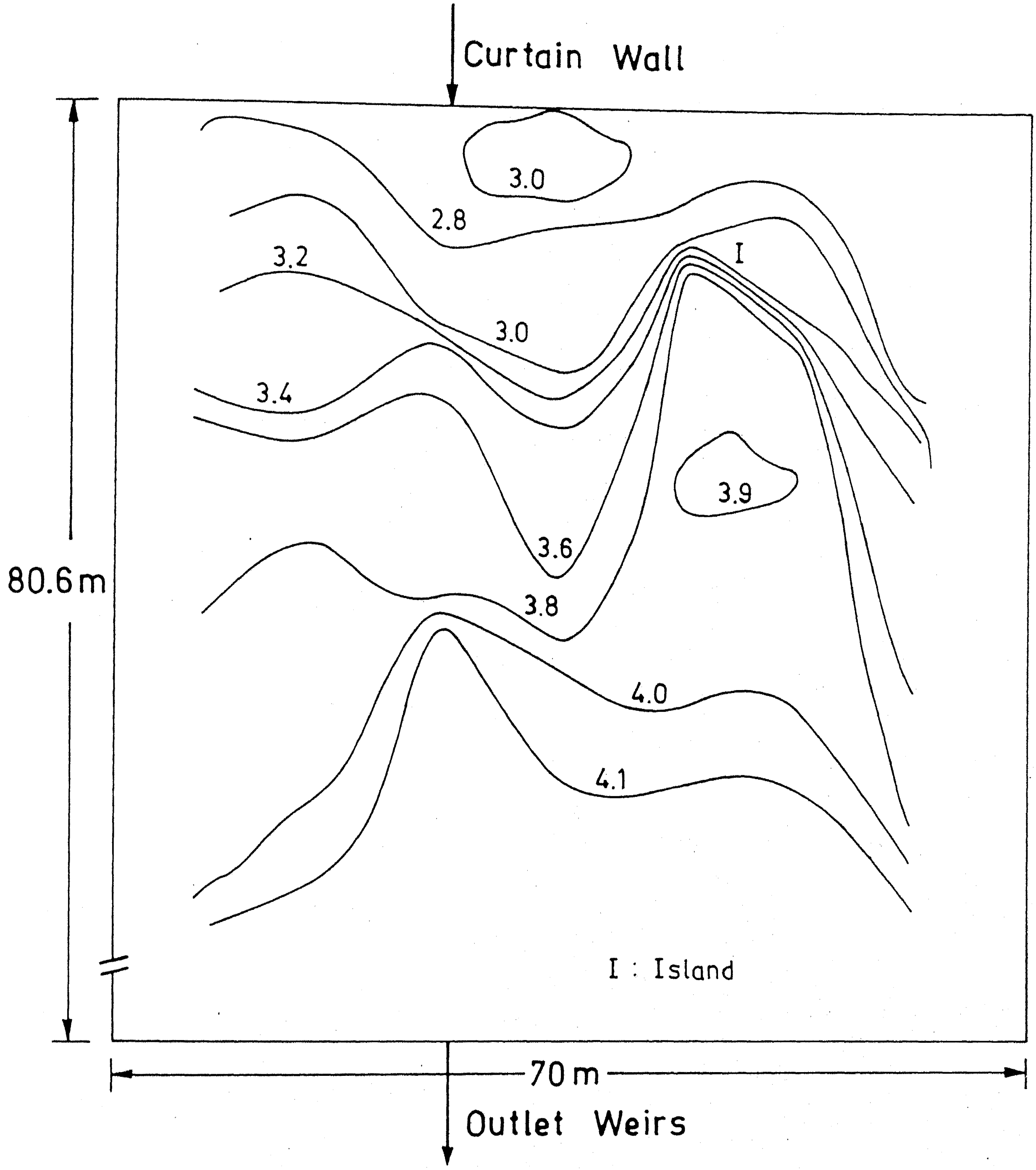


Fig. 6.9. Water Depth Contour Map of Sedimentation Tank No. 1 from Hydrographic Survey.

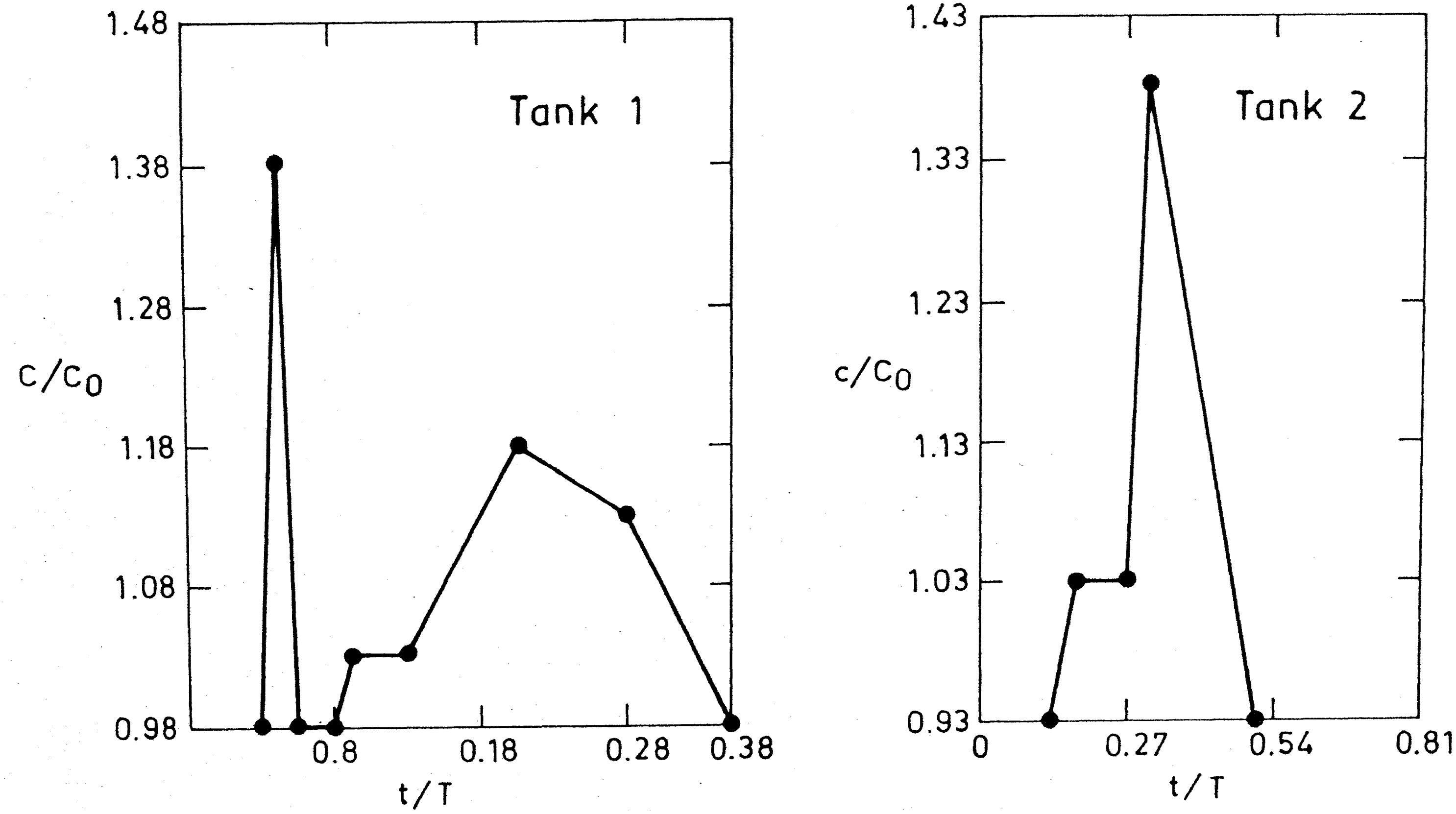


Fig. 6.10. Flow Curves for Sedimentation Tank 1 and 2 Based on NaCl Tracer Studies.

circuiting appears to be due to improper inlet and outlet arrangement. For inlet, only a notch of short length and a curtain wall having perforation at 4.3 m with respect to tank bottom are provided, while water is abstracted from the full width of tank over a rectangular weir.

The values of $[1 - F(t)]$ for different t/T values for both the tanks are calculated and plotted, as shown in Figures 6.11 and 6.12. From these plots the requisite parameters; plug flow fraction; completely mixed fraction and dead space fraction are obtained and presented in Table 6.2.

Table 6.2

Parameters Obtained from Tracer Study for Sedimentation Tanks

	Average detention time, t_a (min)[a]	Plug flow fraction 'p'	Completely mixed fraction '1-p'	Dead space fraction 'm'	Displacement efficiency 't_a/T'
Tank 2	49	37%	63%	61%	32.7%
Tank 1	86	56%	49%	52%	47.8%

The centre of gravity of flow curve gives the flow through time (average detention time), and for sedimentation tank No . 2 and 1, it was found to be 49 and 86 min as against the design detention time of 2.5 hours and 3 hours, respectively. It was due to reduction in volume of the tanks by excessive sludge deposition.

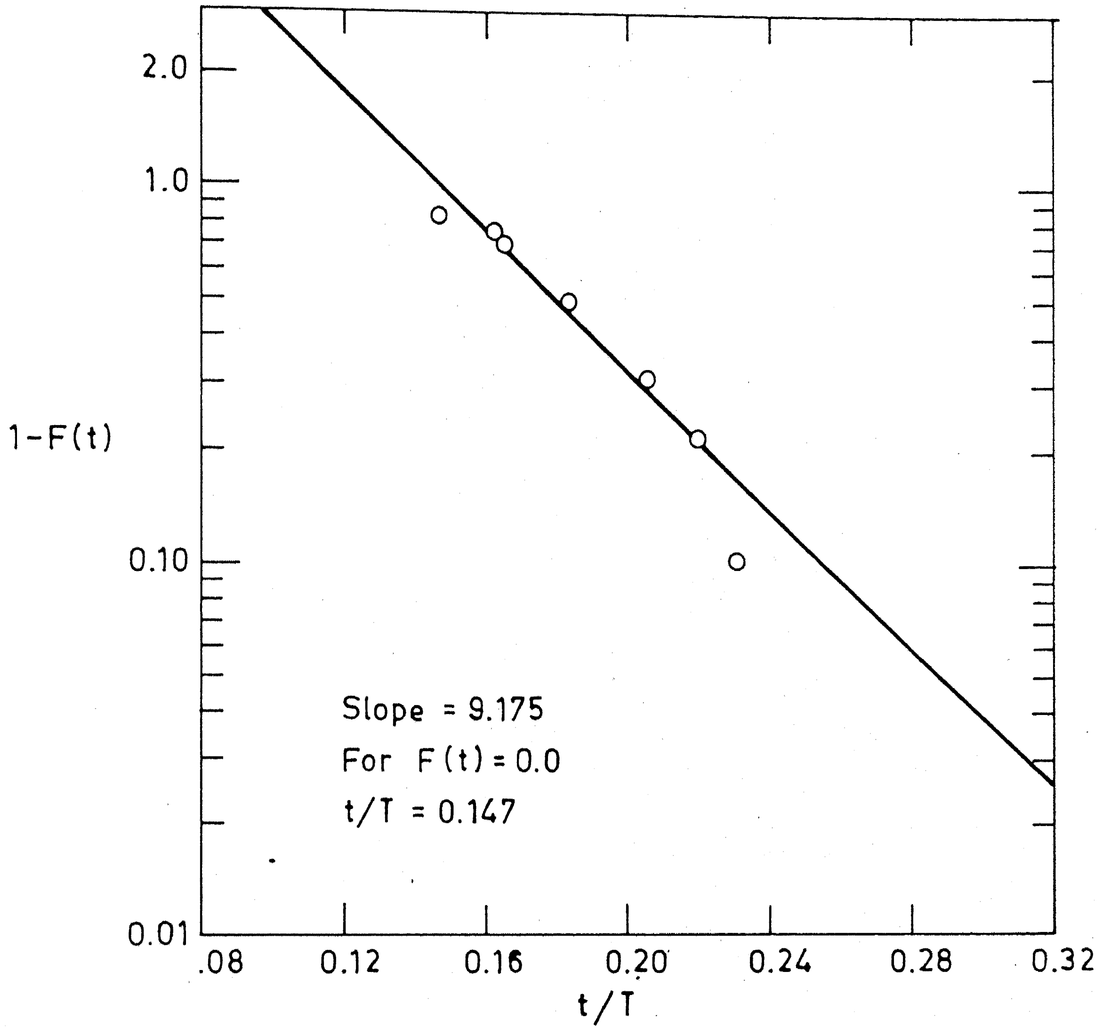


Fig. 6.11. [1-F(t)] as Function of t/T for Sedimentation Tank No. 2.

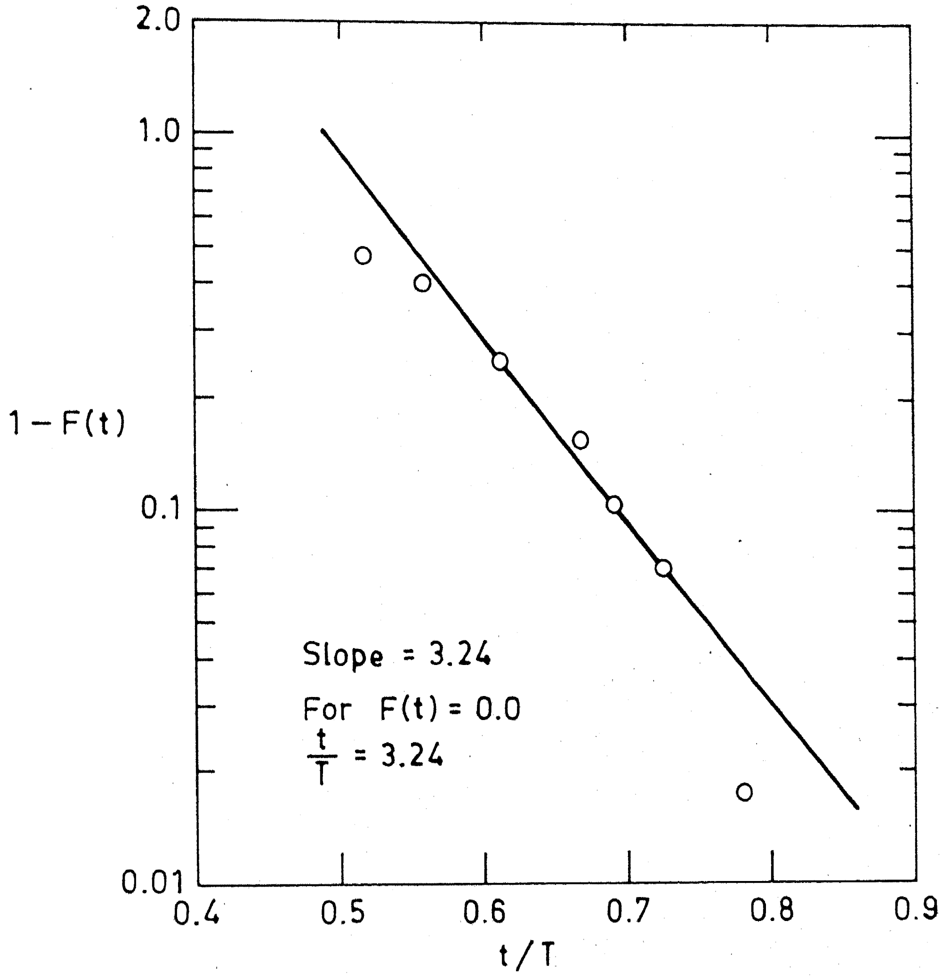


Fig. 6.12. [1−F(t)] as Function of t/T for Sedimentation Tank No. 1.

The dead space fraction (m) indicates the fraction of the effective volume of tank that does not contribute to the turbidity removal. It is noticeable in both the tanks, of course, more with tank No. 2. The excessive sludge deposition in tank No. 2 and improper inlet and outlet arrangements of tank No. 1 appear to be responsible for significant dead fraction. The plug flow fraction 'p' was found to be 37% and 51% for sedimentation tank No . 2 and 1, respectively. This shows a better plug flow condition in tank 1 than tank 2. The completely mixed fraction '1-p' was 63% and 49% for sedimentation tank No . 2 and 1, respectively. Plug flow is highly desirable in settling tanks, where tranquil conditions are needed to promote settling and hence efforts should be made to increase plug flow fraction.

The displacement efficiency of sedimentation tank No . 2 and 1 was found to be 32.7% and 47.8% respectively. As per Cox (1964) the reduction of displacement efficiency below 30% indicates the end of practical utility for a particular sedimentation tank. Hence, it can be said that practical utility of sedimentation tank No. 2 is non existant. However, sedimentation tank No. 1 does not appear to have reached this state.

6.5.3 Turbidity Removal:

In order to have a general view of turbidity variation in different parts of the tank, turbidity values along various cables are presented in Figures 6.13 and 6.14. The rate of turbidity removal is quite high in the beginning

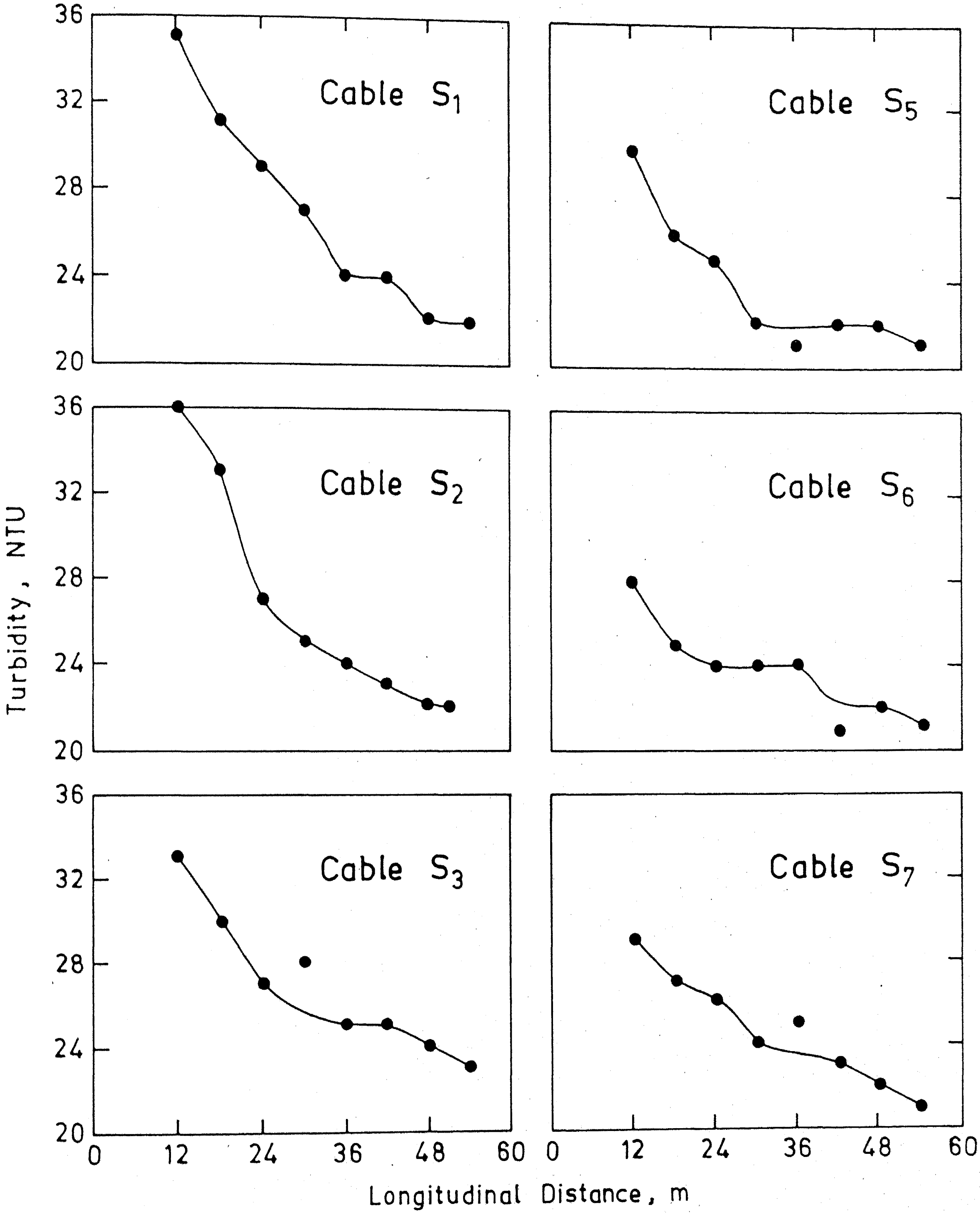


Fig. 6.13. Turbidity Variation Along the Cables in Sedimentation Tank No. 2.

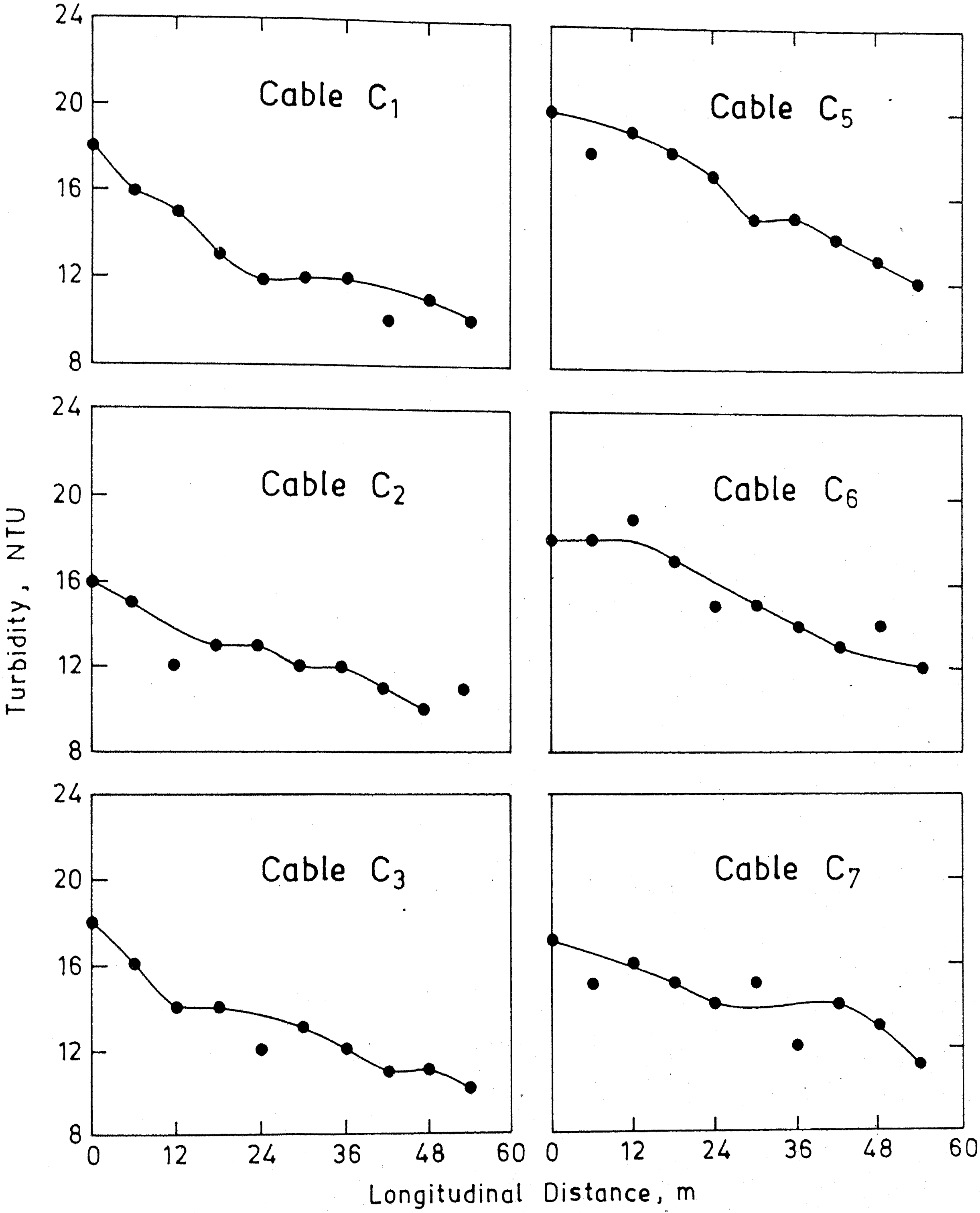


Fig. 6.14. Turbidity Variation Along the Cables in Sedimentation Tank No. 1.

of the tank, however, it subsided in the later portions. At around 48.0m along longitudinal section of tank 1 turbidity is around 11 NTU, while in the clear water channel, which is situated at the end of the tank (80.6 m), the turbidity is 10 NTU.

Turbidity removal upto mid of the tank and, from mid upto the end of the tank has been estimated for a particular set of data and presented in Table 6.3. It is evident from the table that turbidity is mostly removed by the time water travels to the mid length of the tank. The later part of tank does not appear to contribute to turbidity removal.

The data, collected from water treatment plant regarding turbidity removal in the sedimentation tanks during August 1986 to November 1986 covering various ranges of turbidity, show that mean turbidity removals in the high ranges of turbidity (100-600 JTU) were 43.7 $\pm$ 12.6% and 74.9 $\pm$ 10.2% for sedimentation tank No . 2 and 1, respectively, while for low turbidity range (upto 100 JTU), the removals were 22.9 $\pm$ 12.5% and 23.6 $\pm$ 6.0%, respectively. This clearly reveals the poor performance of the tank and need for modification both in design and operation of the tanks.

For an average flow of 60 MLD the overflow rates for sedimentation tank No . 2 and 1 worked out to be 10.4 and 8.5 m/day, respectively. At this overflow rate, particles of diameter more than 11.63 μm should be completely removed in sedimentation tank No. 2. Particles upto 10 μm size 74% and particles upto 5 μm size 18.5%, should be removed. Similarly,

Table 6.3

Percent Turbidity Removals upto Mid Length and Outlet of Sedimentation Tanks

Tank	Cable No.	Turbidity removal upto mid length of the tank '%'	Turbidity removal from mid length to the end of the tank '%'
Sedimentation Tank No. 2	S_1	31.4	12.5
	S_2	36.2	8.7
	S_3	24.3	16.0
	S_5	26.7	4.5
	S_6	18.5	4.5
	S_7	20.7	8.7
Sedimentation Tank No. 1	C_1	33.4	16.7
	C_2	31.3	9.1
	C_3	38.9	9.1
	C_5	19.7	28.6
	C_6	27.8	23.1
	C_7	17.7	28.6

in sedimentation tank No. 1 particles of diameter above 10.52 µm should be completely removed, and particles upto 10 µm size 90.5% and; particles upto 5 µm size 22.6%, should be removed.

It is reported that for a turbidity range of (0-40 NTU) the particle size vary from 2 to 20 µm with a mean of 10 µm (Mohanty, 1976). There should be 74% removal of these particles in sedimentation tank No. 2,while turbidity removal was 39% only. Similarly there should be 90.5% removal of these particles in sedimentation tank No. 1, but the removal of turbidity was only 44.5%. Since the concept of overflow rate has been derived from the theory of ideal settling, quiescent conditions like those in an ideal settling tanks are not expected in a practical sedimentation tank. However, the present removals of turbidity are too low and warrent improvement in design, operation and maintenance of sedimentation tanks.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions:

From the various investigations conducted on the pretreatment units, the following conclusions may be drawn:

1. The actual flow in the unit was only 60 MLD while it was designed for 105 MLD.
2. There was substantial deposition in the flocculators due to which the detention time was reduced to 18.5 min and 18.0 min, for flocculator A and B, respectively, from the design detention time of 30.0 min. Due to this the G values were changed to 37 and 39 sec^{-1}, while the design value was 30 sec^{-1}. The completely mixed fraction was only 42%, also the flow pattern was unstable and complex in the flocculators.
3. There was almost 2.0 m deposition against, the total depth of 2.5 m in the feed water channel. In the end reaches the top row of the perforations (of the perforated wall) were blocked. The flow was not being distributed uniformly across the width and along the depth of sedimentation tanks.
4. The hydrographic survey revealed that sedimentation tank No . 2 and 1 have been silted to the extent 78% and 48% of the tank volumes, respectively. Deposition is more like a ridge in the beginning.
5. The flow through timesof the sedimentation tank No. 2 and 1 were found to be 49 min and 86 min agains' the

detention period of 2.5 hours and 3.0 hours for a flow of 60 MLD due to short circuiting and reduced tank volumes.

6. The flow patterns in both the tanks were unstable and complex with existence of considerable short circuiting. This has led to poor turbidity removal in the tanks. Particularly sedimentation tank No. 2 has a very poor turbidity removal efficiency of 43.7 $\pm$ 12.5% even at higher ranges of influent turbidity.

7. In the effective volume of sedimentation tank No. 2 and 1 the various fractions estimated were, dead space fraction 61% and 52%; plug flow fraction 37% and 51%; and completely mixed 63% and 49%, respectively. The excessive sludge deposition in sedimentation tank No. 2, improper inlet and outlet arrangement of sedimentation tank No. 1 appear to be responsible for significant dead fraction.

8. The displacement efficiencies obtained clearly indicated the practical unutility of sedimentation tank No. 2 which needs desludging

7.2 Recommendations:

Based on the results obtained, the following recommendations are made for better performance of pretreatment units of Allahabad Water Works.

1. As the hydraulic jump used for mixing was weak in nature, the mixing should be intensified by incorporating a spillway of height 1.3 m, the head over the

spillway would be 0.304 m. The design and profile of spillway is given in Appendix B1. This will give a Froude number of 4.94.

2. The velocity gradient which is used in the design is based on jar test, which will not be reproduced in actual practice. Hence, in order to have better flocculation than existing, the velocity gradient in the flocculator should be increased. This will also obviate the deposition in the flocculator and hence completely mixed fraction may also be increased. Also a tapering velocity gradient in the flocculator is desirable.

For these purposes the depth of flow should be increased from 1.2 to 1.8 m and slot width reduced to 0.6 m in Compartment 1 and from 1.8 to 2.45 m in Compartment 2 without any change in slot width. This will give a tapering of G value from 89 sec^{-1} to 21 sec^{-1} and Gt from 3.73×10^4 to 1.67×10^4.

3. For uniform distribution of flow in the sedimentation tanks wooden or metal planks, which would be of the height of sedimentation tanks; able to move in the vertical as well as in the horizontal direction, can be provided. By manipulating the number of perforation (some in the early reaches can be plugged while others in the later reaches can be opened), a uniform flow across the width and along the depth of tank can be ensured.

4) The inlet and outlet arrangement of sedimentation tank No. 1 need to be modified based on model studies.

Following general recommendations are made from the observation of the plant.

1. The sedimentation tanks should be desilted atleast once a year.
2. It is advisable to feed the alum in the form of solution. It may be accomplished by means of a perforated trough or perforated pipes diffuser system in order to distribute it across the stream of water as well as possible.
3. Water levels should be maintained in such a way, so that the flow from the launders into the clear water channel will be free flow.
4. A reduction in alum dose may lead to a better economy.

8. SUGGESTIONS FOR FUTURE WORK

1. It appearsthat occurrence of head loss around a baffle is a complex phenomenon. Both bend loss and expansion loss occur simultaneously. More studies are to be conducted on number of treatment plants, in order to obtain coefficients which would give the net effect of these two losses so that better design is possible.
2. More work is to be done for designing the tapered feed water channel, in terms of the extent of tapering, determination of head loss by section to section and accordingly providing of perforations and their spacing at different sections.
3. Although a lot of research has been done on different types of inlets, still there is very little information on the persistence of inlet disturbance to a particular length and depth of tank with different inlet velocities. Therefore, more work is needed to establish the effective inlet zone, effective settling zone and effective outlet zone in actual sedimentation tanks.

REFERENCES

Argaman, Y.A. (1971). Pilot plant studies of flocculation. Journal of American Water Works Association, 63, p. 775.

Bhole, A.G. and Prasad, V.S. (1973). Physical factors affecting flocculation in water treatment. Journal of Indian Water Works Association, 5, p. 185.

Camp, T.R. (1936). A study of the rational design of settling tanks. Sewage Works Journal, 8, p. 742.

Camp, T.R. and Stein, P.C. (1943). Velocity gradients and internal work in fluid motion. Journal Boston Society of Civil Engineers, 30, p. 219.

Camp, T.R. (1946). Sedimentation and the design of settling tanks. Transactions, American Society of Civil Engineers, 71, p. 445.

Camp, T.R. (1953). Studies of sedimentation basin design. Sewage and Industrial Wastes, 25, p. 1.

Camp, T.R. (1955). Flocculation and flocculation basins. Transactions, American Society of Civil Engineers, 120, p. 1.

Cox, C.R. (1964). Operation and Control of Water Treatment Processes. World Health Organization, Monograph Series No. 49, Geneva.

Dick, R.I. (1981). Sedimentation since Camp. Journal Boston Society of Civil Engineers, 68, p. 199.

El-Baroudi, H.M. (1969). Characterization of settling tanks by Eddy diffusion. Journal of Sanitary Engineering Division, American Society of Civil Engineers, 95, p. 527.

Fair, G.M., Gayer, J.C., and Okun, D.A. (1968). Water and Wastewater Engineering. John Wiley and Sons, New York.

Harleman, D.F. (1961). Stratified flow in "Handbook of Fluid Mechanics" (V Streeter, ed.). McGraw Hill Publication, New York (Cited in Hudson, 1961).

Hayes, F. Ch. and Stuip, J. (1975). Manual for Hydrographic and Hydrometric Survey. Delft University of Technology, Pub. No. 145, p. 114.

Hazen, A. (1904). On sedimentation. Transactions, American Society of Civil Engineers, 53, p. 63.

Hudson, Jr., H.E. and Wolfner, J.P. (1967). Design of mixing and flocculating basins. Journal of American Water Works Association, 59, p. 1257.

Hudson, Jr., H.E. (1981). Water Clarification Processes; Practical Design and Evaluation. Van Nostrand Reinhold Company, New York.

Iqbal, S.M. (1978). Investigation on the performance of hydraulic jump as a rapid mixing device of alum coagulant. M.Tech. Thesis, Indian Institute of Technology, Kanpur, India.

Miyanami, K., Tojo, K., Fujiwara, Y., and Aratani, T. (1982). Effect of mixing on flocculation. Industrial and Engineering Chemistry Fundamentals, 21, p. 132.

Mohanty, P.K. (1976). Inlet and outlet modifications to the existing water settling basins at Kanpur Water Works. M.Tech. Thesis, Indian Institute of Technology, Kanpur, India.

Murphy, K.L. (1964). Tracer studies in circular sedimentation basins. Proceedings of 18th Industrial Waste Conference, Purdue University, Ext. Ser., 115, p. 374.

Rankine, R.C. (1959). Weir on sedimentation tanks. Public Works Journal, 12, p. 66.

Rebhun, M. and Argaman, Y. (1965). Evaluation and hydraulic efficiency of sedimentation basins. Journal of Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, 91, p. 37.

Standard Methods for the Examination of Water and Wastewater (1980). (15th ed.), Jointly published by American Water Works Association, American Public Health Association and Water Pollution Control Federation, New York.

Surya Rao, S. (1980). Design for remodelling of sedimentation tank No. 3, Aishbagh Water Works Lucknow. A consultancy report submitted to U.P. Jal Nigam.

Thirumurthy, D. (1969). A breakthrough in the tracer studies of sedimentation tanks. Journal of Water Pollution Control Federation, 41, p. 405.

Villimonte, J.R. and Rohlich, G.A. (1963). Hydraulic characteristics of circular sedimentation basins. Proceeding of 17th Industrial Waste Conference, Purdue University, Ext. Ser., 112, p. 682.

Weber, Jr., W.J. (1972). Physico-chemical Processes for Water Quality Control. Wiley Interscience, New York.

APPENDIX A

DESIGN OF PRETREATMENT UNITS AS PER REPORT OF U.P.JAL NIGAM

A.1 Design of Flocculator:

Flocculator is constructed in a part of the sedimentation tank (105.6 X 70.0 X 5.3 m). The actual design carried out has been presented here.

The flow in each flocculator is taken as 52.5 MLD. The optimum value of G and t is assumed to be 30 sec^{-1} and 30 min, from jar test. The velocity gradient in the baffled chamber is given by

$$G = 1.34 \times 323 \sqrt{(h_f/t)}$$

where 1.34 is the multiplying factor at 26°C.

The head loss is calculated as 14.41 cm. An average velocity of 15 cm/sec is assumed in the channel (v_1). Therefore flocculator length = 30 X 60 X 0.15 = 270.0 m. The channel area required = $\frac{52.5 \times 10^3}{24 \times 3600 \times 0.15}$ = 4.05 sq.m. Assuming depth of the channel as 2.45 m, the width of the channel = $\frac{4.05}{2.5}$ = 1.653 m.

Total head loss to be attained in the baffled channel is 14.4 cm. Head loss due to friction in the channel is given by

$$Q = \frac{1}{n} A R^{2/3} (h_f/L)^{1/2} \qquad \text{where } n = 0.013$$

which gives head loss due to fraction as 0.195 cm.Assuming velocity at slot (v_2) to be 25 cm/sec, the head loss in the

baffled flocculator with (N-1) number around the end baffles is given as

$$N \frac{v_1^2}{2g} + (N-1) \frac{v_2^2}{2g} + 0.195 = 14.4$$

from the equation N is found to be 33.55. So 34 numbers of channel have been provided, for this v_2 = 24.76 cm/sec, which is between 15 to 45 cm/sec.

Baffle slot width X depth X v_2 = discharge

Or baffle slot width = 1.00 m

Let clear width of the compartment = W metres

Then (W - 1.00) X 34 = 270.0 m

Therefore, W = 8.94 m.

A.2 Design of Permeable Wall:

The permeable wall is as shown in Figure 6.5(A).

Assume depth of permeable wall = 2.50 m

Area of permeable wall = $70.0 \times 2.5 - 2 \times \frac{1}{2} \times 1.76 \times 1.11$

= 173.05 sq. m.

Therefore, area to handle half flow = $\frac{173.05}{2}$ = 86.52 sq.m.

Assuming mean port velocity as 30 cm/sec and head loss coefficient through port as 1.4,

Head loss through port = $1.4 \times \frac{300^2}{2 \times 9810}$ = 6.42 mm

Approaching velocity v_a = $\frac{0.6075}{2.475 \times 1.4}$ = 17.55 cm/sec.

Its kinetic energy = $\frac{17.55^2}{2 \times 9810}$ = 1.57 mm

The head loss through ports (6.42 mm) is about 4 times of the kinetic energy (1.57 mm) due to approaching velocity.

Total port area = $\frac{0.6075}{0.3}$ = 2.025 sq.m.

A port diameter of 20 cm has been selected from a table presented in the report which gives a velocity gradient of 30.8 sec^{-1}. Therefore, 20 cm diameter port 65 numbers shall be provided in each half at a spacing of 0.825 m.

A.3 Design of Settling Zone:

Turbulent entry zone of settling basin

= 10 times the spacing between ports

8.0 m

For a settling velocity of 1.5 cm/min the length of settling zone =

$$\frac{Q}{\text{Average breadth X Velocity}} = \frac{1.215 \text{ X } 60 \text{ X } 100}{1.5 \text{ X } 63.78} = 76.30 \text{ m}$$

Thus total length including turbulent entrance zone =

8.0 + 76.2 = 84.20 m

Length of the basin from the beginning of the flocculator channel to the end of permeable wall =

8.94 X 2 + 0.20 + 1.85 = 19.93 m

The remaining length of 1.47 m (105.6 - 19.93 - 84.2) of the existing basin may be considered as outlet zone in which the flow and remaining suspended particles assemble and are carried out to the effluent conduit.

APPENDIX B

B1. Design of Proposed Spillway:

$$Q = 105 \text{ MLD} = 1.215 \text{ m}^3/\text{sec}$$

The discharge equation for spillway is as follows:

$$Q = \frac{2}{3} C_d \sqrt{2g}\, L\, h^{3/2} \qquad \text{B.1}$$

where h = head over spillway; L = length of spillway; C_d = discharge coefficient.

Assume h = 0.20 m

$$\frac{h}{P} = \frac{0.2}{1.3} = 0.154 \quad \text{where P = height of spillway = 1.3 m}$$

From Engineering Hydraulics, by Hunter Rouse, p. 534,

$$M = 4.0 \qquad \text{where} \quad M = \frac{Q}{Lh^{3/2}}$$

$$h = \left(\frac{Q}{ML}\right)^{2/3} = \left(\frac{1.215}{4.00 \times 1.8}\right) = 0.308$$

$$\frac{h}{P} = \frac{0.308}{1.3} = 0.22, \qquad M = 4.05, \qquad h = 0.304$$

and $\frac{h}{P}$ = 0.233 which is almost same.

$$\text{Approach velocity } V_a = \frac{1.215}{1.8(1.3 + 0.304)} = 0.42 \text{ m/sec}$$

$$\text{Hence} \quad \frac{V_a^2}{2g} = 0.009 \text{ m}$$

$$y_1 + \frac{V_1^2}{2g} = P + h + \frac{V_a^2}{2g} = 1.3 + 0.304 + 0.009 \qquad \text{B.2}$$

$$y_1 V_1 = \frac{1.215}{1.8} = 0.675 \text{ m}^2/\text{sec} \qquad \text{B.3}$$

Solving equations (B.2) and (B.3) by trial and error

$$y_1 = 0.124 \text{ m}, \qquad v_1 = 5.45 \text{ m/sec}$$

$$F_1 = \frac{V_1}{\sqrt{gy_1}} = \frac{5.45}{\sqrt{9.81 \times 0.124}} = 4.94$$

$$\frac{y_2}{y_1} = \frac{1}{2}\left[(1 + 8\,F_1^2)^{1/2} - 1\right] = 6.50 \quad \text{hence } y_2 = 0.806 \text{ m}$$

From Figure 72, p.146 of Elementary Mechanics of Fluids by Hunter Rouse,

For $F_1 = 4.94$, $\frac{H}{y_1} = 6.0$, $\frac{L}{y_2} = 2.3$, $H = 0.75$ m and $L = 1.83$ m

As per U.S.B.R. stilling basin No. 3 given in U.S.B.R. Engineering monograph No. 25, Hydraulic Design of Stilling Basins and Energy Dissipators, the following chute blocks, baffle blocks and solid sills may be provided

h_1 = height of chute blocks = D_1 = 0.13 m

h_3 = height of baffle blocks = 0.917 D_1 = 0.12 m

h_4 = height of solid sill = 0.54 D_1 = 0.067 m

W_3 = width of baffle blocks = 0.75 h_3 = 0.09 m

Spillway Profile:

As per 'Characteristics of Flow Over Terminal Weirs and Sills' by Kandaswamy and Rouse, Journal of the Hydraulics Division, Proc. ASCE, 1957

$$\frac{h_s}{P_s} = \frac{0.304}{1.3} = 0.234, \quad \frac{h_s}{P_s} = \frac{h_w - Z_{max}}{P_w + Z_{max}} = \frac{1 - \frac{Z_{max}}{h_w}}{\frac{P_w}{h_w} + \frac{Z_{max}}{h_w}}$$

Assume $\frac{h_w}{P_w} = 0.25$

From Figure 7 of reference, $\frac{Z_{max}}{h_w} = 0.112$

$$\frac{h_s}{P_s} = \frac{1 - 0.117}{4 + 0.117} = 0.215$$

B.4

which is nearly same as the assumed value.

As we know $h_s + P_s = h_w + P_w$

$$h_w + P_w = 0.304 + 1.3 = 1.604 \qquad B.5$$

Solving equations(B.4)and(B.5)we get

$h_w = 0.32 \qquad P_w = 1.284 \qquad Z_{max} = 0.036$

The dimensionless co-ordinates of the lower nappe of flow over a sharp crested weir are first obtained from the reference and then the dimensional co-ordinates are computed.

x/h_w	Z/h_w	x	Z
0.05	0.045	0.016	0.0144
0.10	0.082	0.032	0.0262
0.25	0.118	0.080	0.0378
0.50	0.064	0.160	0.0205
0.75	-0.045	0.240	-0.0144
1.00	-0.232	0.320	-0.0743
1.25	-0.456	0.400	-0.1460
1.50	-0.741	0.400	-0.2372
1.75	-1.054	0.560	-0.3370
2.00	-1.428	0.640	-0.4570
2.25	-1.840	0.720	-0.5900
2.50	-2.231	0.800	-0.7140
2.75	-2.839	0.880	-0.9100
3.00	-3.446	0.960	-1.1030

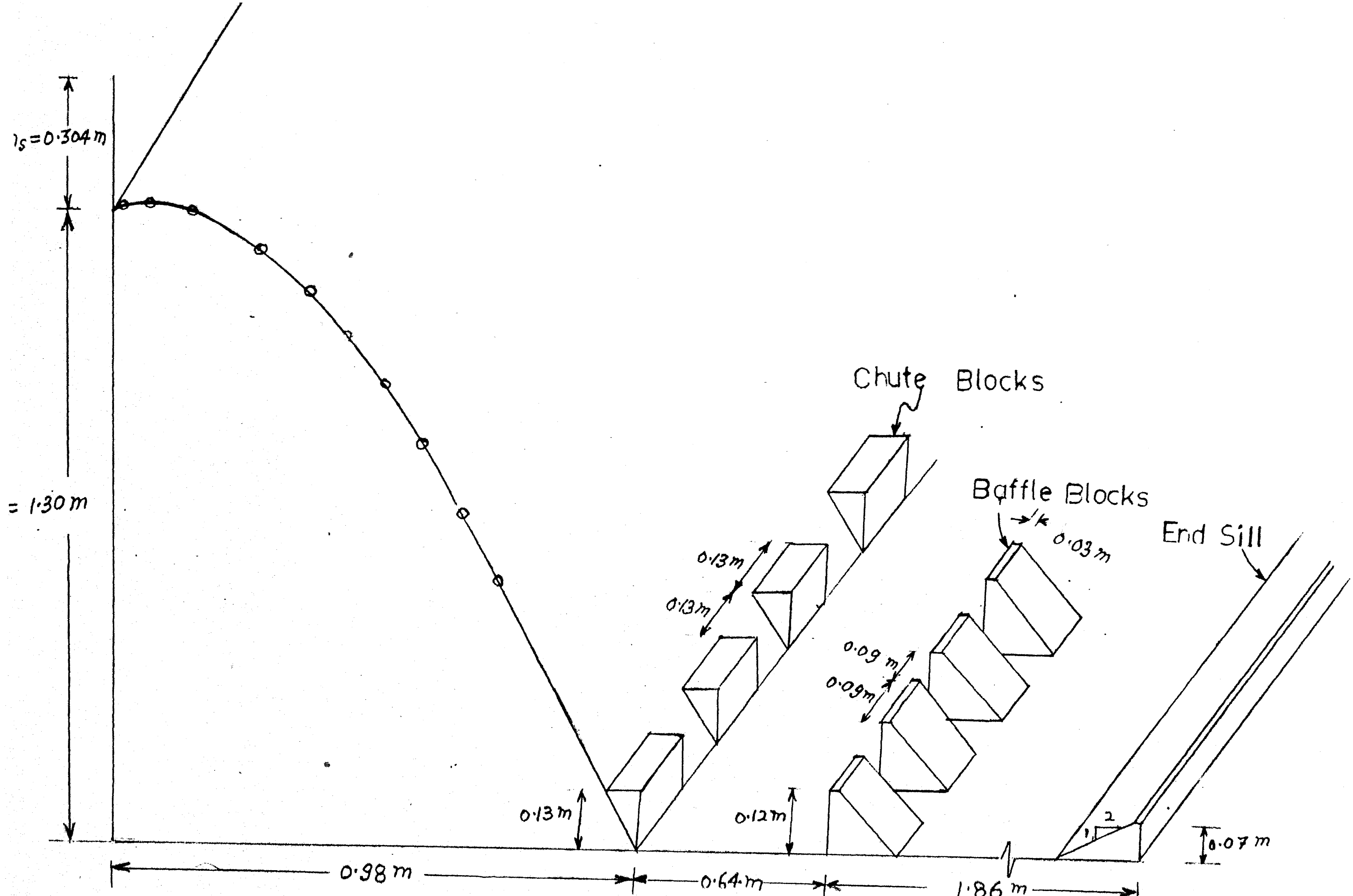
B2. Modification in the Design of Flocculator:

As the flocculator is already existing and the arrangement of compartments and their dimensions are fixed. Tapering of G can only be obtained by varying the depth of flow and slot width in the flocculator. The design has been obtained by satisfying the following equations

$$G = \left(\frac{wh_f}{\mu . t}\right)^{1/2}$$

in which $\mu = 0.8295 \times 10^{-3}$ Kg/m sec at 30°C.

Total head loss in each flocculator's compartment will be as follows



PROFILE OF PROPOSED SPILLWAY

$$h_f = (N-1)\left[\frac{V_s^2}{2g} + \frac{(V_s - V_c)^2}{2g}\right]$$

The detention length is given by

$$V_c t = N(L_c - b_s) + (N-1)b_c$$

where L_c = length of compartment.

The depth of flow is varied from 1.2 to 1.8 m when the slot width is 0.6 m in Compartment 1, in Compartment 2 length of flow is varied from 1.8 to 2.45 m without any change in slot width. The results of the computations are listed below

y (m)	1.2	1.8	1.8	2.45
N	18	18	18	18
b_c (m)	1.6	1.6	1.6	1.6
b_s (m)	0.6	0.6	0.85	0.85
V_c (m/sec)	0.181	0.121	0.121	0.09
V_s (m/sec)	0.483	0.322	0.227	0.167
h_f (m)	0.282	0.125	0.055	0.029
t (sec)	419	626	589	792
G (sec^{-1})	89	48	33	21
G.t	3.73×10^4	3.00×10^4	1.95×10^4	1.67×10^4

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